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Phosphorous Removal Technologies

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Phosphorous Reduction in the Assabet River

A Major Qualifying Project Report:

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ABSTRACT

The goal of this project was to investigate the ability of different technologies to remove phosphorous from wastewater discharged into the Assabet River. Pilot study data from various technologies was obtained and analyzed. From gathered data a full scale Waste Water Treatment Plant was designed incorporating one of the researched technologies.

ACKNOWLEDGEMENTS

This Major Qualifying Project could not have been completed without the knowledge of subject matter from Professor James O'Shaughnessy of the Civil & Environmental Engineering Department at Worcester Polytechnic Institute. A special thanks is extended to Professor O'Shaughnessy for his assistance and patience during the development of the project and the Team.

MQP Design Requirement

This project presents the results of *Phosphorus Reduction in the Assabet River*. To meet the design requirement it was necessary obtained and analyzed data gathered from plan pilot studies on three new phosphorus removal technologies. Pilot studies revealed pros and cons for each technology consisting of cost analysis, maintenance, and performance. After analyzing the three technologies all were designed to accommodate 3 MGD Wastewater Treatment Plant (WWTP). Along with the full scale design of each technology a complete design for a WWTP was also performed. Design for the WWTP include a complete-mix activated sludge system, trickling filters, primary and secondary clarifiers, aeration tanks, and rotating biological contactors. A recommendation for a technology is supplied along with a figure displaying its exact position in the WWTP.

Executive Summary

Increased amounts of phosphorus in any body of water will lead to numerous environmental issues. Phosphorus leads to foul smells and overgrown of algae in turn leading to a poor environment for inhabitants of the water. Our groups' goal was to design a full scale WWTP with the ability of lowering phosphorus less than 0.01 mg/L. To accomplish this goal the team developed a set of objectives.

1. Investigate causes of excess phosphorous found in the Assabet River.
2. Identify three new technologies capable of lowering levels of P in effluent to at least 0.01 mg/L.
3. Design a full scale WWTP with a new technology installed.
4. Make recommendations for installations of researched technologies for WWTP designed in the future.

Additionally, the focus on causes for excessive phosphorus levels in the Assabet River involved conducting background research. This helped identify possible solutions to lowering levels of P in the future placing less strain on the WWTP. If lower levels of P come into the plant fewer renovations need to be made in order to meet new Massachusetts laws. Higher levels of phosphorus in bodies of water have proven to effect most recreational activities such as fishing and swimming. However all who use the river as a source of water should be responsible for its upkeep.

Public awareness about the effects of high phosphorus levels of water is a must. Using Phosphorus free soaps and detergents is one way to reduce ground water contamination. Once overall levels have been reduced it is essential to maintain them. Maintaining these levels can only be accomplished with help from residents of surrounding towns and cities. Fortunately

WWTP's with new phosphorus removal technology will significantly reduce levels and over time allow the river to once again be a good place for recreational use.

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1. Introduction

Phosphorus, a non-metal found in the nitrogen group, is an essential element for the life of living organisms. Forms of the element are denoted as white, red or black phosphorus. White phosphorus is the most harmful form, commonly found in rat poison. Although phosphorus can be harmful, certain applications can be extremely useful to human life. Concentrated phosphoric acids may be used in fertilizers ranging in applications of farm production to lawn care. Phosphates are used in a wide variety of every day life such as sodium lamps, steel production and even in military applications. More commonly, we can fin phosphorus in pesticides, toothpaste and detergents.

Over time humans have changed the natural phosphate supply by adding phosphate-rich manures into soil. Excessive use of phosphate-containing detergents has also enriched water supplies across the country. Food products such as cheese, sausage, and hams all contain phosphorus. A human body containing high amounts of phosphorus can lead to kidney damage and osteoporosis. Similarly, a body deprived of phosphorus also raises health risks.

Phosphorus entering the environment, typically through discharge of wastewater ends up in surface waters. Once in the water, phosphorus does not react with other chemicals quickly. The element finds its way into bodies of organisms and into the soil. This leads to large quantities of phosphate in the environment which typically is not removed properly. Due to the large addition of phosphorus into the environment from human products, the phosphor cycle has been disrupted. Resulting consequences lie in excessive growth of algae, and duck weed. Organisms similar to these use massive amounts of oxygen and prevent any sunlight from entering the

water. Other living aquatic organisms begin to die off due to these unliveable conditions. This process is known as eutrophication.

Currently the Assabet River is experiencing this phenomenon known as eutrophication. Mass DEP has new laws and regulations that restrict levels of phosphorus entering the environment specifically into bodies of water. Laws and Regulations that govern levels of phosphorus can be found in the following chapter. In meeting new regulations set forth by the Mass DEP Local Waste Water Treatment Plants (WWTP's) have had to introduce new technology for purifying water. Specifically, three technologies have emerged supplying claiming the ability to lower phosphorus levels to a new low. In evaluating the Assabet River, a pilot study was conducted at the Marlborough-Westerly WWTP to determine which technology would prove to be most economical.

This project will examine the results of the three technologies, taking into account phosphorus removal along with overall costs associated with installation, operation, and maintenance. After an in depth analysis has been preformed, the team will design a 3.0MGD (million gallons per day) Waste Water Treatment Facility. The designed facility will include one of the previously researched technologies for maximum phosphorus removal. Three disinfection tanks will also be analyzed to determine which is best suited for the needs of the WWTP. Designs calculations for a complete-mix activated sludge system will be performed twice (Design 1 and 2) with different BOD_5 concentrations. One design will be chosen and identified as the recommended plan for construction. Other design calculations include primary and secondary clarifiers, aerated grit chamber, and hydraulic loading rates. All calculations for the design can be found in the Appendix.

The ultimate goal for this project is to lower the levels of phosphorus in the Assabet River to meet the new standards set forth by the Mass DEP. In order to accomplish our ultimate goal I have broken it down into specific objectives. These objectives are as follows:

- Objective 1
    - Determine the cause surrounding high levels of Phosphorus in Assabet River
- Objective 2
    - Review new technologies used in lowering levels of phosphorus.
- Objective 3
    - Review current layout and conditions at the Marlborough-Westerly WWTP located on the Assabet River
        - Average flow
        - Footprint size
        - Levels of phosphorus discharged back into river
- Objective 4
    - Gain access to data recorded for pilot study conducted at Marlborough-Westerly WWTP
    - Analysis of Pilot study
    - Test results
        - Level of phosphorus discharged compared to average daily flow between three technologies
    - Compare costs associated with each individual technology
        - Instillation costs
        - Operational cost

        - Maintenance cost
    - Benefits and drawbacks of each individual technology
        - Upgrading technology
        - Ability to achieve lower levels of Phosphorus for new standards in future years
- Objective 5
    - Determine which technology would perform best for the design parameters of a 3.0MGD WWTP consistently and economically.

This project includes information about lowering the levels of phosphorus multiple ways. All data provided was recorded onsite at the Marlborough-Westerly WWTP in an effort to choose a particular technology to install on site. This data is used to then design a 3.0MGD plant which will achieve the lowest possible levels of phosphorus in an effort to revive the Assabet River.

2. Massachusetts Department of Environmental Protection

The Massachusetts Department of Environmental Protection (MDEP, 2007a), insures proper operation and maintenance for all WWTP. Responsibilities include monitoring all bodies of water and determine which ones are polluted. Once water source has been deemed impaired the DEP is required by the Federal Clean Water Act to develop a plan for revitalization. The developed plan must bring the water source into compliance with current Mass Water Quality Standards. In this developed plan a "pollution budget" is assigned based on the level of toxicity.

When developing this budget, a process referred to as the Total Maximum Daily Load (TMDL), a point source or sources of pollution must be identified. Along with point source(s), non-point source(s) also must be located. Dealing with the Assabet River, an effluent dominated stream, there are four major municipal publicly owned treatment works or POTWs. Three other minor facilities are also located on the river. MDEP (2007b) labelled the Assabet River as an impaired body of water due to the levels of Total Phosphorus being recorded. Under Mass water quality standards, the river has been labelled a Class B. Waters classified as a Class B are capable ones that are capable of providing and supporting habitat for fish and other aquatic life. Class B waters are also capable for primary and secondary recreational uses such as swimming and fishing.

In 1998 the Assabet River was listed by the MDEP (2007b) as having organic enrichment and low dissolved oxygen. Qualities such as these typically refer to the river experiencing a phenomenon known as eutrophication. The phenomenon in bodies of fresh water is generally associated with high levels of phosphorus. To prevent further damage to the water quality and surrounding environment a TMLD for total phosphorus was identified. Meaning the four major POTWs must decrease the levels of phosphorus that are discharged into the river.

Sources of pollutants were identified through six intensive field surveys. Point sources were then found to be the major contributor to the river in 4 of the 6 surveys. When identifying the point sources, four key nutrients were identified.

- Ortho-phosphorus
- Total phosphorus
- Nitrate
- Total Nitrogen

The major non-point source was found to be due to sediment phosphorus flux which generally occurred during low flow periods. However, according to MDEP (2007b) on March 16th and March 27th of 2000, during testing stream flows were relatively high. March 16th experienced 375ft^3/second and March 27th witnessed 250ft^3/second. During these high flow periods the non-point sources were recorded as contributing some of the highest levels of phosphorus (total and Ortho), Nitrate, and Total Nitrogen.

3. Marlborough Westerly WWTP

3.1 Plant Description

The Marlborough Westerly WWTP is an advanced secondary treatment plant. Using the activated sludge process, the plant is designed to treat an average flow rate of 2.89 MGD. Along with treatment of wastewater, the plant is also equipped to accept and treat septage. A key feature to the plant is their ability to remove and treat and dispose of settleable and floatable solids. This helps reduce TSS and dissolved organic material. Other toxins plant is equipped to remove are listed below:

- nitrification of ammonia
- removal of phosphorus
- disinfection of effluent (using chlorine)
- chlorine removal

Removing these toxins all help to preserve and re-build he phosphorus fractured river. Both primary and waste activated sludge are treated with chemicals to ensure elimination of toxins. Sludge is the dewatered on a belt filter press. Below in figure 1 is a 2.2 Meter belt filter press.

Note this press is not located at the Marlborough Westerly WWTP. Presses come in all sizes. Depending on the size different throughputs are generated. According to Bright Technologies (2006) For this model the press can typically run 2000-3000 lbs. of dry sludge (1-2% Anaerobic Digested Municipal WWTP Sludge) per day with a 400gpm sludge pump. The remains is then picked up and sent to the Easterly WWTP to be processed into useful compost. When running under normal operation or typically daily flow, the Marlborough Westerly WWTP is expected to remove at least 90% of TSS and 5-day biochemical oxygen demand (BOD_5) of the incoming wastewater and loads.

Figure 1: Dewatering Belt Filter Press

(Bright Technologies, 2007)

3.2 Wastewater Flow at Marlborough Westerly

Under a normal flow pattern raw wastewater leaves collection systems to be aerated. The collection system consists of two influent sewers from Boundary Street. Sewage waste delivered to the plant is stored in septage storage tanks. Septage tanks pump the contents into the wastewater flow using septage pumps. The wastewater combined with septage runs through an aeration and grit chamber. The aeration and grit chamber filters out and solids such as sand gravel and metal. This procedure is performed by controlling the settling, allowing floatable materials to rise up to the surface. Periodically the chamber is emptied at which point all grit is removed by a vacuum truck. Grit chambers are effective; however materials such as sticks and cloths do not necessarily sink.

Wastewater flow from the grit chamber travel down to a grinder. Sticks and rags that have gotten through the initial grit chamber are not shredded into small pieces. Remains of the floatable materials are not yet removed from the wastewater flow. All the effluent travels into a flow measure device where it is equally divided into three streams. Using gravity the effluent streams empty into three primary settling tanks. Now the solids that were once sticks and rags along with other organic matter settles and is later removed as sludge. Any floating material still present in the primary tank is also removed and dewatered with the sludge.

Remaining in the tanks is known as primary effluent (PE). From these three primary tanks, the PE flows to a weir splitter box. In this box the flow is split into four aeration tanks. Two smaller tanks receive 20% each and bigger tanks receive 30% of the flow. Microorganisms growing in these tanks feed on the any impurities found in the effluent essentially cleaning the water. Since these microorganisms are continuously growing and multiplying it is necessary to remove excess activated sludge. Excess activated sludge is commonly called waste activated sludge. Wastewater contents in the aeration tank are known as "mixed liquor/activated sludge." On average, the mixed liquor takes several hours before passing through the aeration tank.

Mixed liquor is sent into final settling tanks. Here the biological floc settles and is removed. A clear top left behind is known as the supernatant. Recovered sludge from these final settling tanks is labelled return activated sludge. This return activated sludge is pumped from the tanks back to the weir splitter ensuring the quality of the supernatant.

Clear supernatant on top of the secondary clarifiers is referred to as final effluent. Final effluent flows over the top of the clarifier weirs into a chlorine contact tank. A Chlorine solution is used primarily for disinfection. Using sulphur dioxide to treat the chlorinated effluent rendering it nontoxic, it is finally aerated over a concrete barrier back into the Assabet River.

Remains now only consist of thickened sludge left in the gravity thickener. Sludge is mixed together with a polymer filter aid. It is then pumped onto the belt filter presses for dewatering leaving a fairly dry sludge. Dry sludge from the filter press lands on a conveyor belt into a sludge hopper. This dry sludge remains in the hopper until a truck transports it to the Easterly WWTP to be processed into compost. Figure 2 below shows WWTP dewatered sludge in a hopper ready for transport. Any filtrate left over in the filter, located in the press is sent back to the initial wastewater flow stream.

Figure 2: Sludge Hopper

(Energetické strojírny Brno, 2005)

4. Phosphorus Removal

4.1 Effects on Ecosystem

High levels of Phosphorus effects anyone or anything in direct contact with the water supply. In summer months an increase of water born organisms accumulates, such as algae leaving unpleasant odours. Phosphorus along with Nitrogen is the primary cause for algae blooms which results in eutrophication or an increase of chemical nutrients. This increase in organisms results in less oxygen for the bodies of water and in time leads to the slow decay of many fresh water ponds, lakes and rivers.

High levels of phosphorus and nitrogen can be caused by pollution due to a release of sewage effluent and in some cases runoff from lawn fertilizers. Some situations occur naturally in depositional environments. The process of eutrophication typically enhances both growth and decay of plants. Unfortunately this process tends to favour weeds and algae lowering the quality of the water. A typical algal bloom, known as phytoplankton, is one of the many forms of algae developed under these conditions. Phytoplankton is vegetation that disrupts the normal function of the ecosystem. Algae such as phytoplankton can cause lack of oxygen required by fish and shell fish inhabiting the body of water. Along with disrupting those who inhabit the body of water the high levels of phosphorus also affect human life. The recreational use of these rivers, lakes and streams is not uninhabitable. Activities such as hunting, fishing and swimming are no longer available. Drinking water that may come from the body of water is also considered to be contaminated.

In recent years technology for removing phosphorus from municipal waste water has significantly improved. During the 70's, the steps involved in removing phosphorus from wastewater was acknowledged as an essential process. The desire for removing phosphorus can be associated with the new laws requiring lower limits along with the need to please those inhabiting areas that are affected by the water supply.

Phosphorus can be found in multiple forms which can be treated various ways. The major forms of phosphorus found in wastewater are organic phosphates, pyrophosphates, tripoliphosphates, and orthophosphates. In order to remove these multiple forms of phosphorus various technologies exist including methods of Chemical precipitation, Biological assimilation, and Physical filtration. Each of these methods proves to be effective although the conditions surrounding each case can influence the method of removal (Strom. 2006). The challenge is to

obtain low levels of P with minimal cost while not increase the produced sludge immensely. Obtaining low P levels is possible although the challenge of lowering both P and Nitrogen is seemingly more difficult. Some rivers and streams contain more metals and other harmful toxics that must be accounted for. To be able to treat all toxins in municipal waste water a balance of using multiple technologies may be required (Strom, 2006).

4.2 Identifying the Cause

In treating municipal wastewater for phosphorus it is essential to determine reasons relating to the high levels being found. According to Rybicki (2006) there are three causes related to high levels of phosphorus.

- Human excreta (30-50%)
- Detergents (50-70%)
- Industry (2-20%)

Rybicki (2006) also states that estimated percents listed above vary depending on locations. A lot of rivers run through multiple towns and counties such as the Assabet River. The river travels through more populated areas such as Concord before and heads southwest towards more rural locations. Communities containing less industry induced phosphorus have the ability to make changes in order to reduce levels of P found in the water. Using dish and hand soap along other forms of washing detergents that are phosphorus free helps to prevent extra P from entering the water supply. When trying to reduce high levels, the community should not depend entirely on the WWTP. Instead with a collaborative effort the communities using the water can prevent adding unnecessary amounts of toxins to the water. This would lower the burden put on the WWTP. Of course steps must be made to inform the general public about issues surrounding

phosphorus removal. A collaborative effort made by the community and the WWTP would ensure lower levels of toxins found in waste water.

In the 70's steps were made to decrease and eliminate phosphorus from powder detergents. A positive result was found when testing the levels found coming into the WWTP. Sludge per capita in municipal wastewater during late 70's ranged from4.0-5.0 gP/cap a day (Cywiński et al. 1972, 1983, WEF MoP, 1992) compared to a 2.5-3.0 gP/cap per day (Balmer&Hultman 1988) found after a phosphorus ban was introduced only emphasizes the effect a community can have on the waste water (as cited in Strom, 2006).

4.3 Chemical Precipitation

4.3.1 Physical filtration

Treating phosphorus using physical filtration primarily removes particulates or organic solids. One method used in physical filtration is through sand filtration. This may prove to be useful although since only a small percentage of phosphorus found in wastewater is a solid chemical precipitation or even the use of a membrane as a second filter is necessary. According to Strom (2006), if it is assumed that 2-3% of organic solids are phosphorous and an effluent total suspended solid of 20 mg/L represents 0.4-0.6 mg/L of effluent P. Membranes have proven to be extremely efficient for removal of P. Not only does the membrane perform the same task as a sand filter in removing TSS, it can also remove and dissolved Phosphorus typically found in waste water. Since Physical filtration is not effective in removing dissolved Phosphorus, it is closely linked with chemical induction.

4.3.2 Chemical Additives

While physical filtration eliminates TSS including organic P the dissolved forms of phosphorus are still present in the water. The process of adding chemicals to the waste water may be preformed different ways. Chemical additives have been practiced extensively over the years experimenting with various precipitants. Tchobanoglous et al., 2003 states that most often employed are compounds of calcium, aluminium, and iron (as cited in Rybicki, 2006). The main issue revolving around chemical dosing is determining the best point in the treatment process to introduce an additive. Neethling and Gu, 2006 state "chemical addition points include prior to primary settling, during secondary treatment, or as part of a tertiary treatment process" (as cited in Rybicki, 2006). Currently practiced methods including primary precipitation are:

- Primary precipitation
- Simultaneous precipitation
- Final precipitation

Older WWTP use only one primary precipitation point. However this was effective, observations show primary precipitation can decrease biodegradable organic compound concentration rendering it a very meticulous process. Simultaneous precipitation has become the primary process used for introducing chemicals into the treatment process. The process is extremely useful during period s of high and low levels of P. found in the waste water. Final precipitation is a process that is found when laws require suspended solid removal to be efficient before discharging the treated water to protected water. Currently the biggest concern surrounding chemical precipitation is the amount of additional sludge produced by the WWTP. Chemicals such as aluminium sulphate produce less sludge than other chemicals introduced into the process, although the increase is still significant. Aside from these three proven practiced

chemical precipitation methods there are new improved forms using similar ideas from previous process. These new developments in technology include:

- Electrolytical method
- Crystillation
- Magnetic separation
- Adsorption

4.4 Biological Treatments

Currently the following two paths being developed for biological removal of phosphorus:

- Upgrading equipment and process optimization at WWTP
- Phosphorus removal in natural system

4.4.1 Upgrading equipment and Process Optimization

Upgrading equipment at the WWTP is directed towards designing proper condition for better optimization of biological P removal (Barnard 1975, 1983, as cited in Strom, 2006). This idea is only effective is emphasis is placed on obtaining good anaerobic conditions including the presence of easily biodegradable carbon sources. It is also necessary to increase the number of secondary clarifiers to insure the maximum reduction of phosphorus.

4.4.2 Phosphorus removal in Natural system

According to (Rybicki, 2006) phosphorus removal in natural systems introduces the same goal of recycling simple things such as paper and plastic. Otherwise known as biological assimilation this method has been used to remove phosphorus from waste water. Phosphorus is essential for the growth of photosynthetic organisms such as plants, algae and some bacteria. This system is especially good for small local systems. Using activated sludge, certain bacteria

ingest the phosphorus, in some cases up to 95% of total P (Rybicki, 2006). This treatment is good for treatment in ponds and wetlands.

4.5 Blue Water

4.5.1 Applications for Containment Removal

Blue Water provides a number of applications for treating water. Blue (2008), list applications that include but are not limited to Tertiary Wastewater, water reuse, drinking water, Lagoon Systems, and Package plants. As of recent, Blue water released the new system used for Primary Wastewater Treatment. For applications of Drinking water, tertiary Waste water, and Water Reuse, a Centra-Flo™ filter is used in the Blue PRO® process. The new Primary Wastewater treatment system uses a Salsnes filter along with the Blue PRO® and Blue PRO-CEPT® process. This new system was recently tested at Hayden Wastewater Research Facility (HWRF) for its effectiveness in removal of containments.

Various applications listed above, coupled with many others proves to make Blue Water extremely versatile in the field of water treatment. Aside from just treating water, other services such as pilot projects, remediation projects, and treatability studies. The mobile units used for pilot studies and remediation projects range in use from industrial wastewaters to seasonal needs. In the case of the Assabet River, Blue Water had a mobile unit at the Marlborough WWTP to demonstrate an effective level of phosphorus removal when compared to three other technologies under the same conditions.

When dealing with containments, the Blue PRO® system is effective for denitrification, and the removal of Arsenic, Turbidity, algae, metal and phosphorus. Blue (2008) recorded testing phosphorus removal, with the system producing results as low as $0.010\,\frac{mg}{Liter\ P}$. Arsenic levels

tested in drinking water are below $0.010\frac{mg}{Liter}$ and nitrogen levels tested after installation of a denitrification system are below$3.0\frac{mg}{Liter\ N}$. Currently the required phosphorus level in the Assabet River, set forth by the Mass DEP is$0.10\frac{mg}{Liter\ P}$. As discussed earlier, spikes in the system require that technology to hit level lower than required in order to meet Mass DEP laws.

4.5.2 Phosphorus Removal

Using reactive filtration Blue Pro is able to achieve low levels is testing for phosphorus. Blue (2008) describes the system as a moving bed filter allowing a continuous regeneration of reactive filter media. Sand used in the moving filter bed is coated with a hydrous ferric oxide (HFO) allowing waste particles to be separated from the process flow. The HFO coated sand absorbs phosphorus leaving extremely low levels in the effluent. Figure 3 illustrates standard sand compared to the sand coated in Hydrous Ferric Oxide shown in Figure 4.

Figure 3: Regular Filtration Sand

(Blue, 2008)

Figure 4: Hydrous Ferric Oxide Sand

(Blue, 2008)

Blue Pro offers both single and double pass systems which can reach total phosphorus levels below 0.010 mg/L. The systems are offered as either a free standing fibreglass unit or as a concrete installation. Table 1 shows the designed system for various flow rates and foot prints.

Table 1: Modular Blue PRO Systems

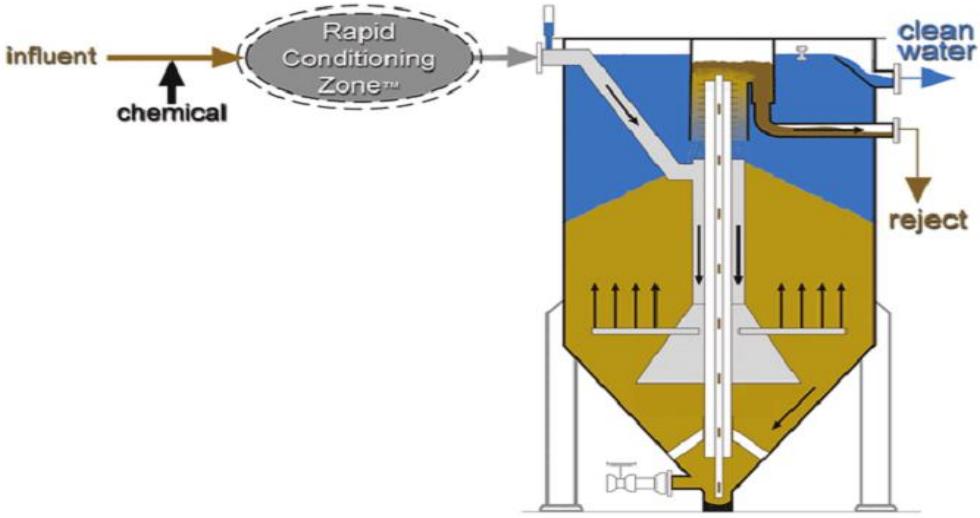
(Blue, 2008)

Blue PRO® Models	Flow Rate	Footprint
Skid systems	5-100 gpm	8′ x 10′ and up
CF-50 fiberglass	0.25 MGD	7′ x 7′
CF-50 concrete	0.25 MGD	7′ x 7′
Quad concrete	1 MGD	15′ x 15′

The double pass system offers a reject cycles which sends bad effluent to an earlier point in the wastewater treatment cycle. This allows for the removal more phosphorus through its secondary system. Below in Figure 5 is a diagram of a Blue Pro filtration system for a wastewater treatment plant. The arrows point up in the sand filter bed shows the recirculation process.

Figure 5: Blue Pro Phosphorus Removal System

(Blue, 2008)



4.5.3 Salsnes Filter

While Blue Water offers many applications in regards to water treatment, the technology of interest is used for Primary Wastewater Treatment. As mentioned above, the Blue PRO® and Blue PRO-CEPT™ process coupled with the new Salsnes filter proves to be extremely effective removing containments.

4.5.4 Hayden Wastewater Research Facility (HWRF)

Blue Water Technologies contracted CH2M Hill to develop the testing and operations plan at HWRF. The testing of the Blue PRO® system involved two tasks for evaluating its effectiveness. Blue (2008) exact tasks are listed below.

- Task 1 – Operational Parameter Testing: This task includes the operation of the Blue PRO™ filtration system under various operating conditions and iron dosages including chemical feed, influent rate, and air rate.

- Task 2 – Long-term Operational Testing: This task includes estimating the reliable effluent total phosphorus concentration that the filters can produce, over a long-term operating period. Records of maintenance and operation are kept during this testing period.

The Hayden Regional WWTP in Hayden, Idaho, includes both primary and secondary treatment, along with disinfection. In the year 2005, the facility treated an average influent flow of 1.2 MGD during with a minimum flow of 0.25mgd and a maximum flow of 2.0mgd. The tested Blue PRO® filtration system involves a chemical addition and a moving bed filtration system manufactured by Applied Process Technologies. A Blue PRO-CEPT™ system will take the rejected stream of wastewater and return it to the front of the WWTP to be re-processed. This system allows the plants operational manager to run a single or dual-pass filtration system.

The basic setup for the Hayden Regional WWTP with contracted Blue Water Technologies is as follows: In Primary treatment the system removes grit before heading to the secondary treatment which includes two 50-ft and one 60 ft-diameter clarifier. Along with the three clarifiers there are two 0.60-Mgal Oxidation Ditches. After Secondary treatment, chlorine is used for the disinfection process. In removing solids, a 0.279g-Mgal Aerobic digester is coupled with a dewatering system before heading to Tertiary Treatment. The Tertiary Treatment of the wastewater was contracted to Blue Water Technologies, Inc. Here two 50-ft^3 filters and three 1,500-gal Chemical Feed Tanks were used, for partial treatment of the effluent.

4.6 Cambridge Water Technology's

4.6.1 CoMag™

The CoMag™ process is designed to remove solids and precipitated particles producing an extremely high quality effluent claimed to be equivalent to ultra-filtration. These systems are

simple cost effective solutions for industrial wastewater. The process involves flocculation and chemical coagulation (Cambridge 2007b). A high Gradient Magnetic Separation system provides extra help when high levels of containments are present. A demonstration of this process is shown in video 1 below.

4.6.2 Benefits of CoMag™

There are many advantages for choosing CoMag™ over other technologies. These advantages are summarized below:

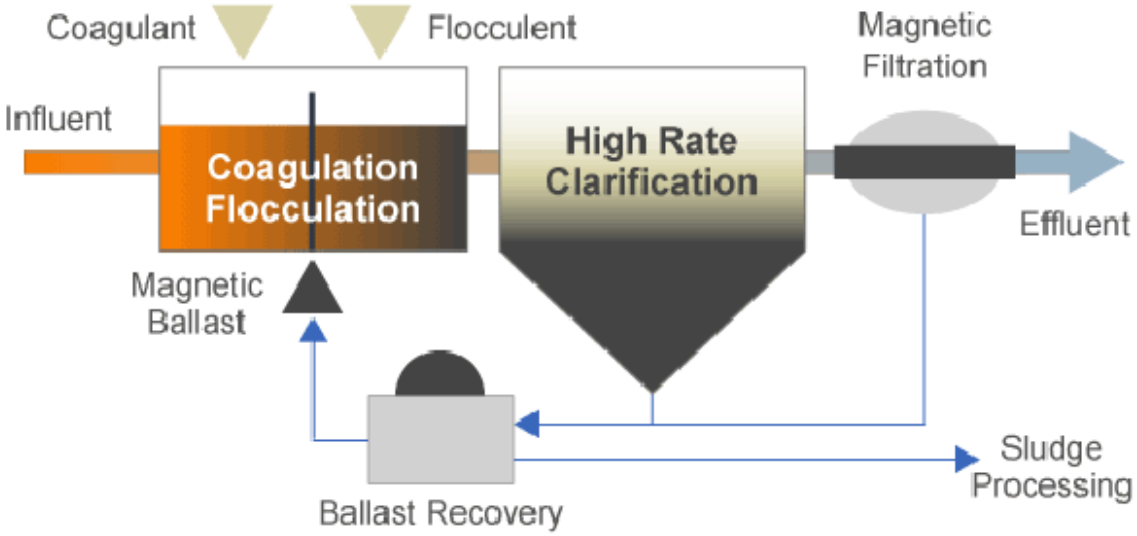
- The cost of construction for one is relatively low due to the use of small solid separation tanks.
- In turn maintenance costs are also low due the lack of inclined plate and tubes which typically require cleaning.
- Once the equipment is installed and operational, little power is consumed and the system only requires moderate chemical use.
- The CoMag™ process is flexible to wide ranges of flows and loads that have minimal effects on the contaminant removal test results.
- Some systems can include a high rate magnetic filter which is a fraction of the size of a sand filter in a DAF unit.
- When chemicals are introduced into the system we can see various results depending on the technology. The CoMag™ system is extremely versatile when it comes to adding various chemicals into the wastewater. The results typically produce a high quality effluent with alum, ferric chloride, ferric sulphate and polyaluminum chloride (PAC).
- The system has a rapid start up in both cold and warm climates, typically 10 minutes.

4.6.3 CoMag™ System

Below in Figure 6 is the setup of a CoMag™ system. As shown, the Magnetic Filtration system demonstrated above is placed before discharging the effluent. This step provides maximum filtration of total phosphorus.

Figure 6: CoMag™ Filtration System

(Cambridge, 2007a)



4.7 ACTIFLO™

4.7.1 ACTIFLO™ system overview

ACTIFLO™ is a conventional water clarification system used in for both drinking water and wastewater. Veolia (2008a) states the compact system uses micro sand as a seed that provides a surface area to form floc. This flocculation then acts as a weight or ballast improving settling characteristics allowing clarifier designs to accommodate high flow rates. These micro sand clarification processes improve the performance and can reduce costs in both primary and tertiary wastewater treatment. The system is extremely efficient for removing TSS, BOD, Phosphorus, COD, metals, fecal coli-form, and other wastewater contaminants.

According to Veolia (2008a) the system has multiple benefits which include a small footprint with a rapid start-up. AcitFLO® also reduces the chemical consumption cutting the cost of operation. Due to the small footprint, minimal and infrequent repairs all decrease operational costs. The flocculation process in the settling tank is used to remove an efficient amount of TSS reducing retention times. ACTIFLO™ is designed for municipal and industrial wastewater for primary treatment before biofiltration and or tertiary treatment for phosphorus and TSS.

4.7.2 Physical-Chemical Process

The ACTIFLO™ process utilizes micro sand ballasted flocculation and lamella settling (Veolia, 2008a). This physical-chemical process induces coagulation which takes place when raw water is sent into the fast mixing tank. The raw water comes into contact with a chemical reagent and a floc is formed. The floc, formed by the coagulated suspended solids then comes into contact with the micro sand which weighs it down. The micro sand then takes the role of a process known as gravitational settling.

Remaining flocculated water flows into the lamella settling zone where they settle on inclined lamellas. Lamella clarifiers contain open channels used in treatment of wastewater and drinking water. They are low in cost and typically 6000 mm in length. Lamella clarifiers are specifically designed with a high projected sedimentation surface with low foot print used for gravimetric separation of solids and liquids. Applications described by Veolia (2008) of lamella clarifiers are as follows:

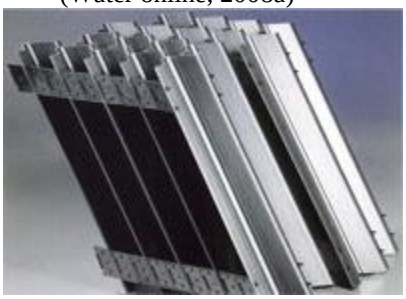
- Drinking water
- Surface water
- Wastewater treatment
- Preliminary

- Aeration for increased suspended solids (SS)
- Sedimentation tanks
- Solid and liquid separation

Lamellas are also extremely versatile when it comes to design. They may be cut into circular forms, and can deliver both open and closed channels. The clarifier is also designed for high temperatures up to 100°C. Figure 7 shows an open Lamella Clarifier used for treatment of wastewater.

Figure 7: Lamella Clarifier

(Water online, 2008a)



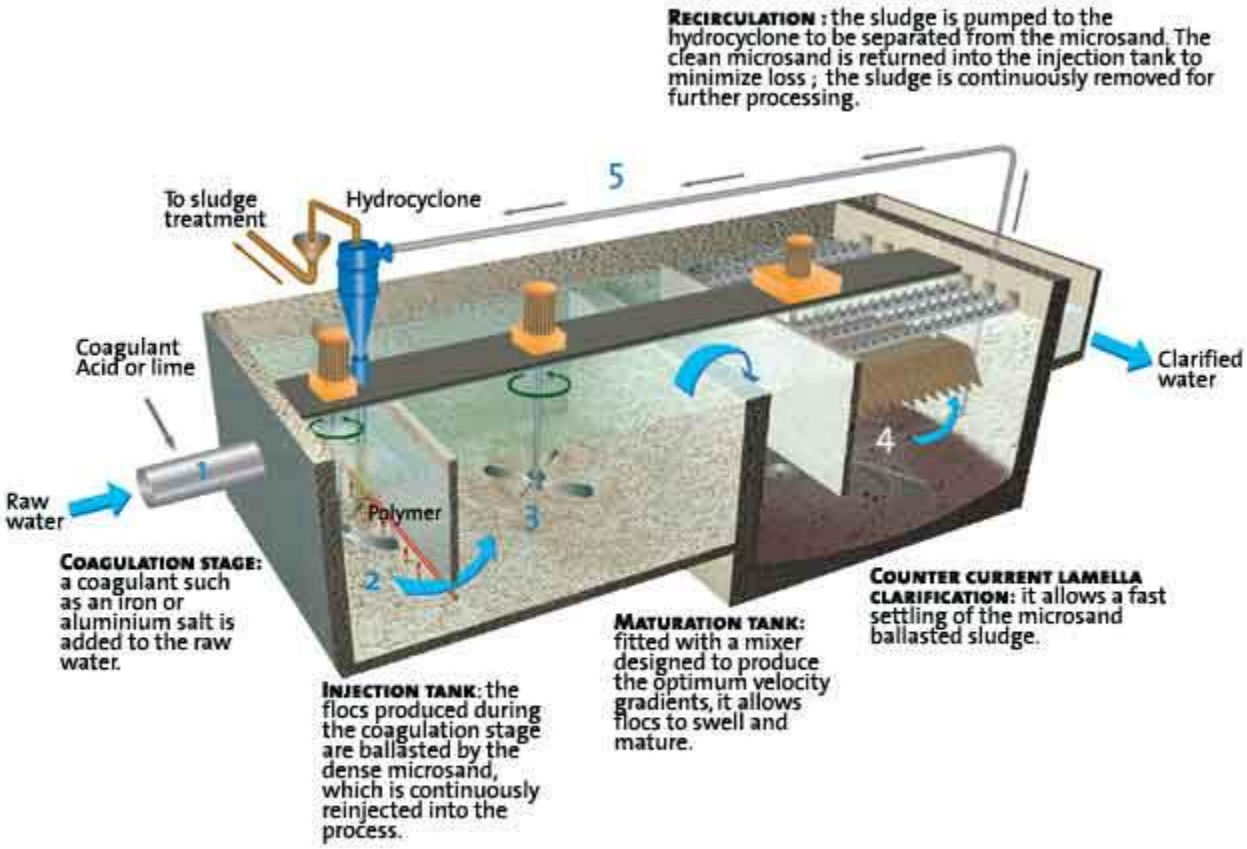
Floc then settles to the bottom of the tank for removal. Removed floc is then sent to the sludge treatment tank. Clarified water is salvaged from the upper half of the settling tank. Any remaining settled solids collected with the micro sand are fed into the hydro-cyclone. This process separates any remaining solids from the sand. In the hydro-cyclone process particles larger than 0.03 to 0.04 mm are fed back into the ACTIFLO™ unit. Generally 3g/m^3 of the micro sand is lost in this process during treatment of wastewater. Figure 8 shows the ACTIFLO™ system as described by Veolia (2008a).

When desired, the process may also be used for grit and grease removal. In order to accomplish this, a vortex degritter would have to be placed upstream to remove any coarse sand 0.3mm or larger. In return the vortex degritter will prevent the build-up of sand in the ACTIFLO™ unit. The ACTIFLO™ unit would also contain a skimmer to remove floating

grease. When dealing with emulsified grease, the process of flocculation would first occur, leading to precipitation of the remains.

Figure 8: ACTIFLO™ system

(Veolia, 2008b)



Due to high-rate settling tanks, a short retention time is all that is necessary. This makes the system able to achieve a high upward velocity. Although applications may vary, typically a 5 minute retention time for coagulation and flocculation with velocities of 80 to 150 m/h can be realised. The size of the unit allows treatment plants to have the system installed with limited

surface area. Veolia (2008a) describes units with flow rates between 500-23,000 m^3/h typically having a diameter of 3.5 to 15 meters.

4.7.3 Performance

Since a coagulant dose is utilized on the ACTIFLO™ system, this generally regulates the performance of the equipment. Table 2 shows removal rates for treatment of combined sere overflow have been listed. When equipped with a vortex degritter, reported removal efficiency is 60% (Stowa, 2006).

Table 2: Sewer Overflow Removal Percents

(Stowa, 2006)

SS	80 – 95 %
COD	60 – 80 %
TKN	15 – 20 %
P	70 – 95 %
Heavy metals	80 – 95 %

When evaluating cost a few variables come into play. The operational stability, cost of maintenance, and capital cost. Typically the operation has been recorded to be similar to other physical-chemical flocculation treatment processes. The advantage to ACTIFLO™ is the quick start up which allows the performance of the system to be table within 15 minutes. Capital costs range from 120 to 200 dollars per m^3/h (however this depends on the specific application, (Stowa, 2006)). Once operational the cost is mainly connected to the coagulant dosage. Stowa (2006) states this dosage is between 30 and 80 mg/l $FeCl_3$ (also dependent on the required level of treatment).

5.0 Design of Wastewater Treatment Plant

5.1 Complete Mix Activated sludge

Activated sludge systems can be designed numerous ways depending on the desired application. For a WWTP designed to lower phosphorus levels released in the Assabet a complete mix activated sludge system is chosen. Design calculations were preformed twice with the following criteria listed in Table 3 below. Note specific values varying between Design 1 and Design 2 highlighted in yellow and green.

Three values in Design two (highlighted in yellow), differing from Design 1 are will significantly change the outcome of both designs. Differing values consist of Mixed Liquor Suspended Solids (MLSS), Mixed Liquor Volatile Suspended Solids (MLVSS), and Mean Cell-Residence Time (θ_d). All three values contribute heavily when determining specific tanks sizes and detention times. The team expects Design 2 will yield larger tank sizes and requirements allowing this system to effectively clean more wastewater thank Design 1.

Table 3: Complete Mix Activated Sludge Design Criteria

	Subscript	Design 1	Design 2
Influent Flow	Q	3.0 MGD	3.0 MGD
BOD_5 (mg/L)	S_o	200 mg/L	200 mg/L
Effluent BOD_5 (mg/L)	E_{BOD5}	20 mg/L	20 mg/L
Temperature (^oC)	T	20 ^oC	20 ^oC
MLVSS	MLVSS	1,840 mg/L	2,200 mg/L
MLSS	MLSS	2,300 mg/L	3,500 mg/L
Ratio	(MLVSS/MLSS)	0.8	0.63
Return Sludge Concentration (mg/L)	----	5,254 mg/L	5,254 mg/L
Mean Cell-Residence Time	θ_c	10 days	24 days
Effluent Biological Solids (mg/L)	$E_{biological\ solids}$	18 mg/L	18 mg/L
Biological Solids % Biodegradable	$\%_{solids\ biodegradable}$	60%	60%
Effluent of Suspended BOD_5 Solids ($N \cdot BOD_L$)	$E_{SS\ BOD_5}$	N = 0.68	N = 0.68
O_2 Consumed per mg	0_2/mg	1.42 mg	1.42 mg

Note: *Influent volatile suspended solids to reactor are negligible
*All MLSS settling data was taken from a plant pilot study located in table 4.
*Design 2 values for MLSS, MLVSS and θ_c are current Massachusetts Waste Water Treatment Plant (MWWTP) operational figures.

Before beginning design the effluent concentration of BOD_5 was determined using the relationship shown in Equation 1 (Metcalf, 1991). Using this relationship the Biodegradable

Portion of Effluent Solids, Ultimate BOD_L of Biodegradable Effluent Solids, and BOD_5 of effluent solids (Equations 2-4, (Metcalf, 1991)) can all be calculated. Calculated values are used to find Influent soluble BOD_5 escaping treatment demonstrated in Equation 5 from Metcalf (1991). Computed values for Equations 2 through 5 for both Design 1 and 2 are listed below in Table 4. Notice that the first four calculated values are not affected by different variable for Design 1 and 2 resulting in exact matches.

Equation 1: Effluent BOD_5

$$E_{BOD_5} = E_{bioliogial\ \ solids} + \mathrm{E_{SS\ BOD\ 5}}$$

Equation 2: Biodegradable Portion of Effluent Solids (BPES)

$$BPES = \%_{solids\ \ biodegradable}\ \ x\ E_{bioliogical\ \ solids}$$

Equation 3: Ultimate BOD_L of Biodegradable Effluent Solids

$$Ultimate\ BOD_L = [BPES](1.42\frac{mg\ O_2 consumed}{mg}cell\ oxidized)$$

Equation 4: BOD_5 of effluent solids

$$\mathrm{E_{SS\ BOD\ 5}} = Ultimate\ BOD_L * (N)$$

Equation 5: Influent soluble BOD_5 escaping treatment (S)

$$\mathrm{E_{BOD\ 5}} = \text{Influent soluble } \mathrm{BOD_5}\ escaping\ treatment(S) + \mathrm{E_{SS\ BOD\ 5}}$$

Table 4: Design 1 & 2 Values

Equation #	Solved Variable	Design 1	Design 2
2	BPES	10.8 mg/L	10.8 mg/L
3	Ultimate BOD_L	15.34 mg/L	15.34 mg/L
4	$E_{SS\ BOD5}$	10.43 mg/L	10.43 mg/L
5	S	9.57 mg/L	9.57 mg/L

5.1.1 Treatment Efficiency E:

Treatment Efficiency is measured by the process, soluble BOD_5 (E_s), and overall efficiency ($E_{overall}$). A generic equation needed to solve for E_s and $E_{overall}$ is illustrated in Equation 6 (Metcalf, 1991). Process efficiency (E_s) from Metcalf (1991), shown in Equation 7 is based on influent soluble BOD_5 escaping treatment (S) found in Equation 5 and influent substrate concentration (S_o) listed in Table 3.

Equation 6: Process Efficiency

$$E = [(S_0 - S)/S]\ x\ 100$$

Equation 7 solves for the efficiency based on soluble BOD_5 (E_s), using the known concentration of 200 mg/L of BOD_5 and previously determined value of S = 9.57 mg/L.

Equation 7: Efficiency based on soluble BOD_5

$$E_s = \frac{(S_o - S)mg/L}{S_o \frac{mg}{L}} x100$$

$$E_s = Soluble\ BOD_5$$

Overall Efficiency, $E_{overall}$ is determined using Equation 8 from Metcalf (1991). Equation 8 similar to 7 is based off the process efficiency formula. Calculation for overall efficiency the effluent concentration of BOD_5 (E_{BOD_5}) is substituted for S. The equation yields a lower percentage than solving for E_s because the overall efficiency takes all forms of BOD_5 into account when comparing to only soluble BOD_5. Values for efficiencies found for both Design 1 and 2 can be found below in Table 5. Note again to this point in design for a complete mix activated sludge system values have not changed.

Equation 8: Overall efficiency

$$E_{overall} = \frac{(S_o - E_{BOD_5})mg/L}{S_o \frac{mg}{L}} x100$$

Table 5: Efficiencies of systems

Equation #	Solved Variable	Design 1	Design 2
1	E_s	95.2 %	95.2 %
2	$E_{overall}$	90.0%	90.0%

5.1.2 Reactor Volume

To determine the reactor volume two equations needed to be combined. Equations 9 and 10 from Metcalf (1991) solve for Mass concentration of micro organisms and hydraulic detention time respectively. Note variables previously not mention listed below.

Equation 9: Mass Concentration of Microorganisms

$$X = \frac{\theta_c Y(S_o - S)}{\theta(1 + k_d \theta_c)}$$

X = mass concentration of microorganisms

K_d = kinetic coefficient

Y= 0.5 lb/lb (Metcalf, 1991, table 8-7 page 394)

Equation 10: Hydraulic Detention Time

$$\theta = \frac{V_r}{Q}$$

V_r = volume of the reactor

Θ = hydraulic detention time (V/Q)

Combining these two equations yields a formula to find the reactor volume V_r shown in Equation 11.

Equation 11: Volume of Reactor

$$V_r = \frac{\theta_c QY(S_o - S)}{X(1 + k_d \theta_c)}$$

Notice Equation 11 from Metcalf (1991), the first difference between Design 1 and 2 appears. Variable X, or MLVSS and mean cell-residence time (θ_c) were larger in Design 2. With a known value of V_r, hydraulic loading volumes may be calculated using Equation 12 (Metcalf, 1991). Note conversion factor of 7.48 gallons/ft^3. Table 6 below shows changed variables between Design 1 and 2 respectively along with calculated volumes for the reactor.

Equation 12: Hydraulic Loading Volume

$$V_{hyrdaulic} = \frac{V_r}{7.48 \; {gallons}/{ft^3}}$$

Table 6: Reactor Volume Comparison

Variable	Variable notation	Design 1	Design 2
MLVSS	X	1,840 mg/L	2,200 mg/L
Mean Cell-Residence time	θ_c	10 d	24 d
Volume of Reactor	V_r	0.97 million gallons	1.27 million gallons
Hydraulic Loading Volume	$V_{hydraulic}$	12968 ft^3	169786 ft^3

It is evident that Design 2 requires a larger Volume. Note that installing larger reactor one would encounter higher costs however when designing for worst case scenario a larger volume would prove to be more efficient and effective during high flow seasons. Along with a larger tank volume a longer mean cell-residence time ensures the highest quality effluent from the Activated sludge system. Based on that assessment the volume of the reactor, V_r will be designed to hold 1.27 million gallons.

5.1.3 Quantity of sludge wasted every day:

Unfortunately not all effluent can be cleaned, purified and re-released into the Assabet River. Sludge, another form of waste must also be taken into account. Sludge is generated from solids that are filtered out through various stages of the treatment process. Equations 13 through 15

from Metcalf (1991) are basic formulas used to calculate the quantity of sludge wasted every day.

Observed yield is affected by the mean cell-residence time (Equation 13). Due to a longer θ_c for Design 2 expect a lower yield. The observed yield also effects all calculations determining the amount of sludge wasted per day.

Equation 13: Observed Yield

$$Y_{obs} = \frac{Y}{1 + k_d \theta_c}$$

$$= \frac{0.5}{(1 + 0.06\ x\ 10)}$$

$$= 0.3125$$

- Increase of mass of MLVSS or P_x:

Design one's observed yield or $Y_{obs} = 0.3125$. This value is then gives the ability to determine the Mass Increase of MLVSS or P_x in Equation 14. Note the conversion factor located under Equation 13 converting mg/L to lbs/gallon. Again the value of P_x allows Equation 15 to be calculated solving the Mass Increase in MLSS or $P_{x(ss)}$. Note in Equation 15 that the values of P_x and (MLVSS/MLSS) ratio differ for design 1 and 2. The varied results lie below in Table 7.

Equation 14: Mass increase of MLVSS (lb/day)

$$P_x = Y_{obs}\, Q(S_o - S)(8.34)$$

Conversion factor*8.34 = conversion factor [lb/Mgal·(mg/L)]

Equation 15: Total Mass Increase of MLSS (lb/day)

$$P_{x(ss)} = \frac{P_x}{(MLVSS/MLSS)}$$

Equation 16: Sludge Wasted (lb/day)

$$Mass\ wasted\ =\ increase\ in\ MLSS\ -\ SS\ lost\ in\ effluent$$

Table 7: Sludge Waste Values

Equation #	Variable	Variable Notation	Design 1	Design 2
12	Observed Yield	Y_{obs}	0.3125	0.205
13	MLVSS increase	P_x	1488.92 lb/day	976.74 lb/day
14	MLSS increase	$P_{x(ss)}$	1,861.2 lb/day	1,550.38 lb/day
15	Mass Wasted	---	1,411.92 lb/day	1,100.02 lb/day

Above listed values reinforce the previous decision of V_r. Weights of wasted sludge found in Design 2 are significantly less than those of Design 1 as expected. Equation 16 from Metcalf (1991) demonstrates how to solve for the wasted sludge (Lbs/day). As previously mentioned the Reactor Tank will be more expensive to construct although, producing less sludge proves to be cost effective. Sludge must be properly dewatered and disposed requiring numerous costs. These costs come from equipment need to dewater sludge, maintenance on machines, disposal location, and the cost of trucking sludge for disposal.

5.1.4 Sludge wasting rate

Sludge wasting rate, if wasting is accomplished from the reactor if the Volatile Suspended Solids (VSS) in effluent is equal to 80% of the Suspended Solids. Separation unit flow or Q_e is assumed equal to Flow (Q). Metcalf (1991) uses Equation 17 to solve for flow rate of water containing wasted biological cells, denoted Q_w. Obtaining a value for Q_w then allows the team to solve for the separation of unit effluent concentration of microorganism, denoted X_e (Equation 18 (Metcalf, 1991)). Values for both designs can be found in Table 8.

Equation 17: Sludge Wasting Rate

$$\theta_c = \frac{V_r X}{Q_w X + Q_e X_e}$$

- Q_w = flow rate of liquid containing wasted biological cells
- $Q = Q_e$ = separation unit flow rate
- X_e = separation unit effluent concentration of microorganism

Equation 18: Separation Unit Microorganism Concentration

$$X_e = E_{bioliological\ \ solids} * (\%\ VSS\ in\ effluent)$$

Table 8: Wasted Biological Cell Liquid Flow Rate Comparison

Variable	Notation	Design 1	Design 2
Mean cell-residence time	θ_c	10 days	24 days
Reactor volume	V_r	0.97 million gallons	1.27 million gallons
MLVSS	X	1,840 mg/L	2,200 mg/L
Separation unit effluent concentration	X_e	14.4 mg/L	14.4 mg/L
Separation unit flow rate	$Q = Q_e$	3 MGD	3MGD
Liquid flow rate w/ wasted biological cells	Q_w	0.078 Mgal/day	0.027 Mgal/day

5.1.5 Recirculation ratio writing a mass balance around the reactor:

Determining a proper recycling ratio for the wastewater a mass balance is preformed around the reactor. To perform a mass balance values for VSS concentration, return VSS concentration must be calculated. Using Equations 19 & 20 from Metcalf (1991) these values are obtained for both designs. Quantities produced for both designs from Equations 19 and 20 are located in Table 9.

Equation 19: VSS Concentration

$$VSS\ concentration\ =\ Return\ sludge\ concentration\ (mg/L)\ \ x\ (MLVSS/MLSS)$$

Equation 20: Return VSS Concentration

$$Return\ VSS\ concentration\ =\ Return\ Sludge\ Concentration\ x\ (MLVSS/MLSS)$$

Performing a mass balance with values for MLVSS, Flow (Q), VSS, and Return VSS is shown in Equations 21 (Metcalf, 1991). Equation 20 solves flow rate for the reactor, Q_r. This flow rate is then used again in Equation 22 from Metcalf (1991) for determining the recycle ratio, α. Actual design values entered for both designs along with their resultant quantities are located in Table 10.

Equation 21: Mass Balance Equation

$$(X\ )\cdot\ (Q+Q_r)=(Q_r\ *\ Return\ VSS)$$

Equation 22: Recycle Ratio, α

$$\frac{Q_r}{Q}=\alpha$$

Hydraulic detention times for the reactor may also be obtained for both designs. Metcalf (1991) uses Equation 23 below to determine hydraulic detention time (θ). Values for both designs were calculated and recorded in Table 9.

Equation 23: Hydraulic Detention Time (θ)

$$\theta = \frac{V_r}{Q}$$

Table 9: Recycle Ratio/Hydraulic Detention Times Comparison

Variable	Notation	Design 1	Design 2
MLVSS	X	1,840 mg/L	2,200 mg/L
Flow	Q	3 MGD	3 MGD
Return VSS	---	4,203.2 mg/L	3,310.02 mg/L
Reactor flow rate	Q_r	2.34	5.95
Recycle Ratio	A	0.78	1.98
Hydraulic Detention Time	Θ	7.76	10.16 hours

5.1.6 Oxygen requirements based on ultimate carbonaceous demand, BOD_L

Oxygen requirements for the reactor tank are governed by the ultimate BOD_L mass of all incoming wastewater. In Equation 24 from Metcalf (1991) this mass is calculated. Note: conversion factor (8.34 mg/L the assumption previously mentioned in Table 3 stating BOD_5 = 0.68 BOD_L. After obtaining a Mass of BOD_L the oxygen requirement per day can be calculated. Metcalf (1991) uses Equation 25 to illustrate lb of O_2/day requirements. Table 10 displays these calculated values for both Designs.

Equation 24: Mass of BOD_L

$$Mass\ of\ BOD_L\ utilized = \frac{Q[(S_o - S)mg/L]}{0.68}\ x\ 8.34$$

- $8.34\ =\ conversion\ factor\ [lb/Mgal \cdot (mg/L)]$

Equation 25: O_2 Requirements (lbs/day)

$$lb\ of\ \ O_2/d = \frac{Q(S_o - S)x\ 8.34}{f} - 1.42(P_x)$$

- $f\ =\ conversion\ factor\ for\ BOD5\ to\ BODL\ (0.68)$

Table 10: Oxygen Requirements

Variable	Notation	Design 1	Design 2
Ultimate Mass of BOD_L of incoming Wastewater	Mass BOD_L Utilized	7,006.7 lb/day	7,006.7 lb/day
Oxygen requirement per Day	Lb O_2/day	4,892.43 lb/day	5,619.73 lb/day

Ultimate BOD_L mass from incoming wastewater does not change between the two Designs. With equal values of ultimate BOD_L mass, solving for O_2 requirements in Equation 25 only contains one variable. The value of MLVSS(P_x) differs between designs thus yielding different O_2 requirements. Design two contained more MLVSS resulting in the need for more O_2 per day. However the tank in Design 2 has a larger volume in turn requiring more O_2.

5.1.7 Food to microorganism (F/M) ratio and volumetric loading factor

Each tank requires microorganisms that feed on bacteria in the wastewater. These microorganisms are released in specific amounts known as Food to Microorganism ratios (F/M). This ratio is calculated using Equation 26 from (Metcalf, 1991).

Equation 26: Food to Microorganism Ratio (F/M)

$$\frac{F}{M} = \frac{S_o}{\theta X}$$

The resultant value of Equation 26 is measured in days, (d). Volumetric loading (Equation 27) from Metcalf (1991) is measured as a factor of time, specifically days. Values for Designs are located in Table 11. Note conversion factor constants listed below.

Equation 27: Volumetric Loading

$$\frac{lb}{10^3 ft^3} \cdot d = \frac{S_o Q}{V_r} \cdot 8.34 \cdot (\frac{1{,}000}{10^3})$$

- $7.48\ {gallons}/{ft^3}$
- $8.34 \frac{lb}{Mgal} \cdot \left(\frac{mg}{L}\right)$

Table 11: Volumetric Loading & F/M Values

Variable	Notation	Design 1	Design 2
Hydraulic detention time	Θ	0.32 d	0.42 d
Reactor Volume	V_r	97,000 gallons	1,270,000 gallons
MLVSS	X	1,840 mg/L	2,200 mg/L
Influent amount of BOD_5	S_o	200 mg/L	200 mg/L
Food to Microorganism	F/M	$0.34d^{-1}$	$0.22d^{-1}$
Volumetric loading	lb $BOD_5/10ft^3 \cdot d$	385	29.47

After computing both volumetric loadings, there is a noticeable difference between the Designs. Design 2 contains a larger reactor volume and requires more O_2 although; it requires fewer microorganisms than Design 1. Both designs contain identical numerators in Equations 26 and 27. Denominators of both equations are where the variables vary between Designs 1 and 2. A higher influx of O_2 into the tank allows the microorganisms to thrive thus requiring a lower F/M ratio. Another benefit to a lower volumetric loading rate for microorganism relates to sludge. Fewer microorganisms' results in less waste sludge to be discarded.

5.1.8 Volume of air required.

Determining the volume of air required for the tank Oxygen transfer efficiency is assumed to be 80%. Blower motors will be designed with a safety factor of 2 to account for periods of high flow. Higher flow periods require the motors to send more air in turn creating more work. Equation 28 illustrates how to determine theoretical air requirement for air containing 23.2% O_2 by weight (Metcalf, 1991).

Equation 28: Theoretical Air Requirement

$$Theoretical\ air\ requirement = \frac{(lb\ of\ \ O_2/d)}{(0.075\frac{lb}{ft^3}x0.232)}$$

$$\frac{4{,}892.43\frac{lb}{d}}{\left(0.075\,{}^{lb}/_{ft^3}\right)(0.232)} = 281{,}174\,{}^{ft^3}/_{d}$$

For air requiring 8% transfer efficiency Equation 29 is used (Metcalf, 1991). Metcalf (1991) uses the resultant of Equation 29, multiplying the result by a safety factor yielding the design air requirement (Equation 30).

- Air requirement at 8% transfer efficiency

Equation 29: 8% Air Requirement

$$Air\ requirement\ at\ 8\% = \frac{Theoretical\ air\ requirement}{transfer\ efficency}$$

Equation 30: Design Air Requirement

$$Safety\ Factor\left(Air\ requirement\ at\ 8\%(^{ft^3}/_{min})\right) = 4{,}882\ ^{ft^3}/_{min}$$

Table 12: Air Requirements

Variable	Design 1	Design 2
Theoretical air requirement	281,174 ft^3/day	322,973 ft^3/day
Air requirement at 8%	2,441 ft^3/min	2,803.6 ft^3/min
Design air requirement	4,882 ft^3/min	5,607.2 ft^3/min

Above Table 12 displays values computed for both designs 1 and 2 from Equations 28 through 30. It is evident that Design 2 yielded higher requirements. Higher air requirements are caused by a greater volume of SS found in the effluent. Although the required values of air are greater and might require a larger motor to produce, the production and results manufactured from Design 2 will surpass those of Design 1. Design 2 displays the ability to efficiently produce more effluent and treat wastewater containing higher concentrations of volatile solids.

5.1.9 Volume Check

Metcalf (1991) states volumes must be checked for air requirements, per unit volume (Equation 31), and per lb $B0D_5$ removed (Equation 32). The volume checks were performed and values for Design 1 and 2 can be found in Appendix B and C respectively.

Equation 31: Air Requirement per Unit Volume Check

$$Air\ per\ unit\ volume = {Air\ requirement\ at\ 8\ \%}/{Q}$$

Equation 32: Air Requirement per lb of BOD_5 Removed Check

$$Air\ requirement\ per\ lb\ of\ B0D_5 = \frac{Air\ requirement\ at\ 8\ \%}{(S_o - S)(Q)(8.34)}$$

5.1.10 Required Recycle Ratio

A required recycle ratio for maintaining mixed-liquor suspended-solids concentrations. This ratio can be determined using Equation 33 from Metcalf (1991). Metcalf (1991) shows how to use a mass balance written around the reactor as seen in Equation 34. Assuming the influent suspended solids (X_o) is equal 0 and $Q_r = (\alpha \cdot Q)$ Equation 34 is rewritten to Equation 35 (Metcalf, 1991), producing another mass balance. After performing a few algebraic steps Equation 36 from Metcalf (1991) yields a value for α. The value of α is the recycle ratio for various rates of under flowing suspended solids. Conditions for Design 1 and 2 are found in Tables 13 and 14 respectively.

Equation 33: Recycle Ratio

$$Ratio = \frac{MLVSS}{(MLVSS/MLSS)}$$

$$\frac{1{,}840\ mg/L}{0.8} = 1{,}472\ {mg}/{l}$$

Equation 34: Mass Balance on Influent to Reactor

$$Q(X_o) + Q_r(X_u) = (Q + Q_r)\ x\ \left[\frac{MLVSS}{(MLVSS/MLSS)}\right] {mg}/{l}$$

Equation 35: Mass Balance on Influent to Reactor with Assumptions

$$\propto QX_u - \propto \left[\frac{MLVSS}{(MLVSS/MLSS)}\right] Q = Q\left[\frac{MLVSS}{(MLVSS/MLSS)}\right]$$

Equation 36: Recycle Ratios at Various Underflow Conditions

$$\propto = \frac{\left[\frac{MLVSS}{(MLVSS/MLSS)}\right]}{X_u\ {}^{mg}/_{l} - \left[\frac{MLVSS}{(MLVSS/MLSS)}\right] {}^{mg}/_{l}}$$

Table 13: Design 1 Recycle Ratios

X_u, mg/l	2,000	3,000	3,500	4,000	5,000
X_u - 1472 (mg/l)	528	1,528	2,026	2,528	3,528
α	2.78	0.96	0.73	0.58	0.42

Table 14: Design 2 Recycle Ratios

X_u, mg/l	4,000	5,000	6,000	7,000	8,000
X_u – 3,492 (mg/l)	508	1,508	2,508	3,508	4,508
A	6.87	2.32	1.39	0.99	0.77

5.2 Aeration Tank

Effluent leaving Primary Clarifiers flows into a wier splitter to evenly distribute wastewater into three Aeration Tanks. According to Ophardt (2003), "the treatment known as activated sludge, microorganisms and wastewater in various stages of decomposition are mixed, aerated, and maintained in suspension." Aeration tanks utilize biological treatment to clean primary clarifier effluent. Biological treatment consists of multiple different microorganisms that feed on

remaining suspended solids. Aeration tanks require a "delicate balance of food and oxygen, are commonly referred to as the mixed liquor suspended solids (MLSS) or activated sludge" (Ophardt, 2003). Resulting effluent contains a "settleable floc" produced by microorganisms (Ophardt, 2003). This "settleable floc" is then removed in secondary treatment (Ophardt, 2003).

To optimize sludge collection a complete-mix activated sludge process, previously designed will be installed immediately after the Primary Clarifiers, preceding the Aeration tanks. Adding this system prior to Aeration tanks will produce a floc with better settling properties. Using Table 16 below the team was able to determine detention periods, (hours), Activated return sludge percentages, and removal efficiencies.

Table 15: Aeration Tank Design Standards

(O'Shaughnessy, 2008)

	BOD Loading				
	(Lbs BOD_5/day) $1000ft^3$	(Lbs BOD_5/day) Lb of MLSS	Period (Hours)	Average Return Sludge (%)	BOD Removal Efficiency (%)
Conventional	30-40	0.2-0.5	6.0-7.5	30	95
Step Aeration	30-50	0.2-0.5	5.0-7.0	50	90-95
Contact Stabilization	30-50	0.2-0.5 6.0-9.0**	0.5-1.0*	100	85-95
Extended Aeration	10-30	0.05-0.2	20-30	100	85-95
High Rate	100+	0.5-1.0	2.0-4.0	100	85-90
Pure Oxygen System	120+	0.5-1.5	1.0-3.0	30	90-95

Note: *Aeration period for the first aeration tank
**Aeration period for the second tank with the return activated sludge

The WWTP will use convention aeration detention periods for design. Detention periods listed in Table 16 highlighted in yellow shows of 6.0-7.5 hours. The team has decided to calculate using a 6 hour detention period. Detention periods, listed in hours are then converted into days yielding 0.25day. With the WWTP have three trains' results in three aeration tanks. Knowing the Flow (Q) = 3.0 MGD, with each train having a Q = 1.0 MGD, the hydraulic loading volume is determined using Equation 37. With a resulting value labelled in gallons, dividing the conversion factor 7.48 gallons/ft^3 yields a Hydraulic Loading Volume in ft^3. A value of 250,000 gallons was converted to 33,442 ft^3.

Equation 37: Hydraulic Loading Volume

(O'Shaughnessy, 2008)

$$Hydraulic\ loading\ volume\ =\ (Q)(Detention\ period)$$

After the loading volume is determined organic loading must be found. Equation 38 illustrates values and function required to establish organic loading. Values used in the equation consists of a 200 mg/L BOD_5 concentration, Q = 1.0 MGD and the conversion factor 8.34 lbs/gallon to yield a resultant value in Lbs BOD_5/day.

Equation 38: Organic Loading (Lbs BOD_5/day)

$$Lbs\ BOD_5/_{day} = conc.(^{mg}/_{l})\ x\ Q\ x\ 8.34\ ^{lbs}/_{gallon}$$

With organic loading in Lbs BOD_5/day calculated for total incoming effluent before any clarification, it will be assumed that the Primary Clarifier removes 35%. With the Primary Clarifier removing 35% the Aeration tanks will receive 65% of all remaining BOD_5 loadings.

Equation 39 shows proper calculation for this value. Once Aeration Tank loading has been determined, we refer back to Table 16. Highlighted in green is a range of 30-40 Lbs BOD/1000ft^3. We have chosen to use 35 Lbs BOD/1000ft^3 for it is between low and high values

of the range. This chosen value will be used to calculate the volume in ft^3 which will then be converted to gallons using conversion factor of 7.48 gallons/ft^3 (see equation 40). With an Aeration Tank Volume of 20,143 ft^3 being less than the Hydraulic Loading Volume (33,442 ft^3) the design is satisfactory. The Aeration Tanks Volume was calculated to be 150,669 gallons. For design purposes the tank volume is rounded up and will be constructed to hold 160,000 gallons. Equations 39 and 40 were taken from O'Shaughnessy (2008).

Equation 39: Aeration Tank BOD_5 Loading

$$\frac{Lbs\ BOD_5}{day}(1.0 - 0.35)$$

Equation 40: Aeration Tank Volume

$$Volume = \frac{\left(Aeration\ Tank\ loading\ {}^{lbs}/_{day}\right)}{35\left[\frac{\left({}^{Lbs\ BOD_5}/_{day}\right)}{1000 ft^3}\right]}$$

5.3 Clarifier Surface Area

5.3.1 Primary

Solids found in wastewater typically is 30% suspended and 5% colloidal and 65% dissolved (Water, 2007b). The function of Primary treatment is to remove suspended solids. Both inorganic and organic debris settle in the tank thus separating settle able and suspended solids from effluent. Effluent leaving primary clarifiers is mainly composed of dissolved and colloidal solids (Water, 2007b).

Primary Clarifiers (P.C.) are based on influent flow rates and overflow rates. Primary clarifiers will be designed with a 600 gallon/day –ft^2 overflow. Equation 41 illustrates using flow and overflow rates to determine proper surface area.

Equation 41: Primary Clarifier Surface Area

O'Shaughnessy (2008)

$$Surface\ Area = \frac{Flow}{Oveflow\ Rate}$$

5.3.2 Secondary

Secondary Clarifiers are used for additional removal of organics. Secondary treatment uses biological treatment methods. Microorganisms are grown in a controlled environment to feed on remaining solids. Mainly feeding on colloidal and dissolved solids, the microorganisms use organic matter to survive (Water, 2007b).

Designing Secondary clarifiers, similar to a primary are also based on flow and overflow rates. Secondary clarifiers require a recirculation therefore adding another variable demonstrated in Equation 42. Recirculation ratios were determined during design of the Trickling Filter. A recycle ratio of 1 and Flow (Q) equaling 2.0 MGD is calculated in section titled Trickling Filter. Using this ratio in Equation 42 surface area for the Secondary Clarifier is found to be 3,334 ft^2.

Equation 42: Secondary Clarifier Surface Area

(O'Shaughnessy, 2008)

$$Surface\ Area\ = {Flow + Recirculation}/{Overflow\ rate}$$

5.4 Trickling Filter

The Trickling Filter develops a slime layer on the surface containing numerous populations of microorganisms. Wastewater passing through a trickling filter develops this slime layer consisting of multiple organisms, bacteria, and protozoa (Water, 2007a). Thickness of the surface slime layer increases overtime. Layers with increased depth prevent oxygen from

penetrating the surface. With time, the thickness of the slime layer increases preventing oxygen from penetrating the full depth of the slime layer. Solids sloughed into Trickling Filters have anaerobic cores (Water, 2007a). Soon anaerobic decomposition begins to occur at the surface of the media due to lack of oxygen.

Design of the treatment plant includes primary clarifiers preceding the trickling filters. Allowing clarifiers to remove to collect scum, derbis, and suspended solids the Trickling Filter can operate properly by maintaining the hydraulic load (Water, 2007a). Trickling Filter effluent typically consists of colloidal, dissolved and suspended solids. Both organic and inorganic matter can be attached to filter medium leaving the filter. Another possibility of matter escaping in effluent includes living microscopic organisms. Follow trickling filters, effluent containing escaping matter flow to Secondary clarifiers.

When designing trickling filters organic loading must be calculated. Assuming primary clarifiers remove 35% of BOD_5, remaining 65% (0.65) enters Trickling Filter. Three trains for the 3 MGD WWTP require 3 Trickling Filters. Each filter receives a Flow (Q) = 1 MGD. Again the conversion factor of 8.34 will be used. Equation 43 demonstrates how to determine proper organic loading in lbs BOD_5/day.

Equation 43: Trickling Filter Organic Loading

(O'Shaughnessy, 2008)

$$Lb\ BOD_5/day\ =\ (S_o)(8.34)(Q)(0.65)$$

Table 16: Trickling Filter Design Standards

(O'Shaughnessy, 2008)

	Low Rate	High Rate	Two Stage
	BOD_5 Loading		
Lbs/100 ft^3-day	5-25	25-45	45-65
Lbs/acre-ft-day	200-1,100	1,100-2,000	2,000-2,800
Hydraulic loading (MG/acre-day)	2-5	10-30	10-30
gpm/ft^2	0.03-0.6	0.16-0.48	0.16-0.48
Operation	Intermittent	Continuous	Continuous
Recirculation ratio	0	0.5-3.0	0.5-3.0
Depth of Bed (ft)	5-7	5-7	5-7

Organic loading approximately equal to 1,085 lb BOD_5/day gives the ability to design the Trickling Filter. Table 16 contains three design standards for BOD_5 loading (lbs/day) highlighted in yellow. 1,085 lb BOD_5/day, approximately 1,100 BOD_5/day is acceptable for both Low rate and High rate Tricking Filter design. Assuming the possibility for higher loading rates during winter and spring months and High Rate Trickling Filter will be designed.

After determining Trickling Filters will be designed for High Rate flows, the volume of the tank must be calculated. To calculate the tank volume and BOD_5 loading rate (lbs/day-ft^3) must be selected. A range of 25-45 lbs BOD_5/day-100 ft^3 is highlighted in green (Table 16). With 1,100 BOD_5/day, this design will use a 30 lb BOD/day-1,000 ft^3. Calculating Tank Volume may not be accomplished using Equation 44.

Equation 44: Tank Volume

(O'Shaughnessy, 2008)

$$Volume = {Organic\ Loading}/{lbs\ BOD_5/day - 1{,}000\ ft^3}$$

Next a selection for depth must be chosen. For High Rate Trickling Filters depths from 5-7 ft are acceptable. Depth values are located in Table 16 are highlighted in Pink. Area of the tank will was calculated three times using depths of 5, 6, and 7 ft. Equation 45 calculates Area for tank with a depth of 5 feet where Equation 46 uses a conversion factor of 43,560 ft^2/acre converting value to acres. Equations 45 and 46 were retrieved from O'Shaughnessy (2008).

Equation 45: Tricking Filter Area (5 ft Depth)

$$Area = {Volume}/{Depth}$$

Equation 46: Ft^1 converted to Acres

$$= {Area}/{43{,}560\ ft^2/acre}$$

Recirculation of wastewater may not be required. For High Rate Filters hydraulic loading must be in the range of 10-30 MG/acre-day. To check loading without recirculation refer to Equation 47 from O'Shaughnessy (2008).

Equation 47: Hydraulic Loading (w/out recirculation)

$$Hydraulic\ loading = {1.0MGD}/{acre} = \frac{MG}{acre - day}$$

With a range to low for High Rate Trickling Filters recycle ratios must be used. These ratios are listed below.

- With a recycle ratio of 1, flow is 2.0 MGD
- With a recycle ratio of 2, flow is 3.0 MGD

Performing Equation 47 again with recycle ratio flow rates yields 11.76 mg/acre-day (ratio 1), and 17.64 mg/acre-day. With a flow of 2 MGD, meeting the requirements set for by design standards recycle ratio 1 will be selected for design.

5.5 Rotating Biological Contactor

Rotating Biological Contactors (RBC's), using a rotating disc enables the process to be a fixed-film biological reactor. Discs, typically plastic (non-metallic) are mounted on a shaft in the tank acting as a surface for bio-mass to grow (Water, 2007c). Discs rotate half in the wastewater between 2-4rpm. Rotating at a slow rate allows microorganisms to collect forming a film. This film, in some cases several millimeters thick, is use for removal of organics and absorption of oxygen. Removal percentages can be upwards of 95% for surface loading rates of 2-3 gpd/ft^2 (Water, 2007c). The rotating discs provide adequate aeration and mixing using minimal power proving to be extremely efficient.

Design for RBC's is based on organic and hydraulic loading. However organic loading is more important than hydraulic loading. Table 17 shows Organic Loading Design Standards for RBC's.

Table 17: Rotating Biological Contactor Design Standards

(O'Shaughnessy, 2008)

Organic Loading	
Overall loading	2.0 lbs BOD_5 / 1,000 ft^2-day
First stage loading	No more than 5.0 lbs BOD_5 / 1,000 ft^2-day
Hydraulic loading	1.0 gal / ft^2-day

The RBC will consist of three stages. Stage one will account for ½ calculated area where stages two and three will each make up ¼ of the total area. The RBC with primary clarifier inherits 1085 lb BOD_5/day. Design for organic loading with and overall loading is illustrated in Equations 48. First stage loading, Equation 49 from O'Shaughnessy (2008), is compared to the overall loading. Comparing the two computed values, overall loading provides a square footage. Values for overall loading and first stage loading were rounded up for design. All loading stage values for the WWTP can be found below in Figure 9.

Equation 48: Surface Media (ft^2)

$$Surface\ media\ = Volume\ \left[\frac{lb\ BOD_5}{day}\right] \Big/ \frac{2\ lb\ BOD}{day\ -\ 1000 ft^2}$$

$$= 543{,}000\ ft^2 \cong 550{,}000\ ft^2$$

Equation 49: First Stage Loading

$$First\ stage\ = \frac{1085 lb\ BOD}{day} \Big/ \frac{5\ lb\ BOD}{day\ -\ 1000 ft^2} = 217{,}000\ ft^2$$

Figure 9: Stage 1-3 Area (ft^2)

$$Total\ Area\ = 550{,}000\ ft^2$$

$$1st\ stage\ = 250{,}000\ ft^2$$

$$2nd\ stage\ = 150{,}000\ ft^2$$

$$3rd\ stage\ = 150{,}000 ft^2$$

6.0 Phosphorous Removal Design Selection

6.1 Pilot Study Cost Estimates

Estimating the cost of each technology is imperative for deciding which will be used in design of the WWTP. In order to estimate a cost for the technologies many factors had to be tested during the pilot study. Variables such as electricity consumption, chemical precipitation costs, operational costs, and capital costs all needed to be accounted. Along with these costs a 20 year cost projection will also be calculated to compare overall life cycle expenses. Chemical cost depended on how many chemicals and how much of each were used to precipitate the wastewater in order to reached levels less than or equal to 0.01 mg/L P.

Along with costs other selection criteria will be considered in the final design. Selection criteria include performance, flexibility, and ease of operation. After reviewing data from Westborough's WWTP pilot study the selection of a technology for use in the WWTP will be made.

Table 18, 19, 20 and 21 gives actual values obtained from the Westborough Wastewater Treatment Facility. Table 18 shows a technical summary for a proposed full scale design including two trains running at 100% each. Note flows for the design WWTP are not equal to those used in the Westborough Treatment Facility pilot study. Therefore these costs can not accurately portray proposed costs for the WWTP our team designed. However these values are a good indicator of differences, both pros and cons between the three technologies which are useful in making a selection in the final design.

Table 18: Proposed Full Scale Design Data

(NEWEA, 2008)

Description	Blue PRO®	CoMag™	ACTIFLO™
Clarification System			
Flocculation Time (min)	----	10	6
Loading rate (gpm/**ft**^{2})	3.5	4.4	16.8
Area required (**ft**^{2})	300	175	50
Waste Production			
Clarifier waste (gpm)	375	160	140
Percent of Total waste (%)	7.0	3.0	2.6
Solids concentration (%)	0.03	0.8-1.0	0.1-0.5

As mentioned above, chemical use is critical in estimating a capital cost. Table 18 portrays values of four chemicals used to precipitate phosphorous in the pilot study. It can be seen that Blue PRO® used the most chemicals and ACTIFLO™ the fewest. Table 19 displays the amounts of chemicals each technology consumed during the pilot study in Westborough. Costs for specific amounts of chemicals used (shown in Table 19) are displayed in Table 20. Table 19 most obviously represents chemical usage in specific amounts (mg/L), but it also establishes the ability for each technology to utilize multiple chemicals. Reading the table in terms of flexibility we can conclude that CoMag™ and ACTIFLO™ both have the ability to uses multiple chemicals.

Table 19: Chemical Use Analysis

(NEWEA, 2008)

Chemical		Blue PRO®	CoMag™	ACTIFLO™
Ferric Chloride	Dosage (mg/L)	40	30	30
	Daily Usage (mg/L)	653	490	490
Sodium Hydroxide	Dosage (mg/L)	35	25	25
	Daily Usage (mg/L)	373	267	267
Polymer	Dosage (mg/L)	--	3.0	0.3
	Daily Usage (mg/L)	--	200	20
Aluminium Sulphate	Dosage (mg/L)	--	50	80
	Daily Usage (mg/L)	--	622	995

As mentioned above Table 20 depicts costs for amounts of chemicals used to precipitate wastewater. Along with chemical costs, Labour and Electric consumption costs have also been estimated. Although Blue PRO® uses the most chemicals; electrical costs are significantly less than its competitors. However expensive nature of chemicals total operation and maintenance costs still amount to be the highest.

Capital costs associated with each technology, also listed in Table 20 vary immensely. Blue PRO® is estimated to have the lowest capital cost. This cost could be associated with its expensive O&M expensive. Although a cheaper capital cost is often appealing, when calculating for a 20 year life cycle for each technology, chemical costs will add up quickly. Costs projecting 20 years can be found in Table 21.

Table 20: Operation and Maintenance Costs (O&M)

(NEWEA, 2008)

Description	Blue PRO®	CoMag™	ACTIFLO™
Ferric Chloride	$179,000	$134,000	$134,000
Sodium Hydroxide	$409,000	$292,000	$292,000
Polymer	---	$73,000	$7,000
Consumables	$3,000	$20,000	$1,000
Electrical	$19,000	$24,000	$30,000
Labour	$3,000	$14,000	$1,000
Cost (O&M)	**$613,000**	**$557,000**	**$465,000**
Capital Costs (Millions of dollars)	**$1.622**	**$2.275**	**$1.871**
Total (O&M + Capital)	**$2,235,000**	**$2,832,000**	**$2,336,000**

Analyzing Total Life Cycle costs from Table 21 ACTIFLO™ comes in a strong first for being the most cost effective technology. Surprisingly Blue PRO® with a relatively cheap capital cost coupled with extremely high chemical costs beats out CoMag™ by less than $100,000. With these expenses being estimated values we can assume that Blue PRO® and CoMag™ with relatively close costs can be considered equal. Clearly if a town or city was searching strictly based on cost ACTIFLO™ would be the obvious choice.

Table 21: Twenty Year Cost Analysis

(NEWEA, 2008)

Description	Blue PRO®	CoMag™	ACTIFLO™
	Cost in Millions of Dollars		
Capital Costs	$1.622	$2.575	$1.871
20 Year Life Cycle O&M Cost	$9.808	$8.912	$7.440
Total Life Cycle Cost	**$11.430**	**$11.487**	**$9.311**

Perhaps the most important results from the pilot study conducted are results. Each technology did meet the 0.1 mg/L Total Phosphorus Limit using Ferric Chloride ($FeCl_3$), Aluminium Sulfate (Alum), or Poly-aluminium chloride (PAC). When dosing wastewater with $FeCl_3$ it may be noted that optimum concentrations varied between technologies. A compiled range of $FeCl_3$ for the three technologies was between 20 and 35 mg/L. Again optimum concentrations varied between technologies for Alum between 40 and 80 mg/L. Even when flow conditions were varied all three technologies did meet the required 0.1 mg/L Total P limit.

After conducting these tests the pilot study also was interested in findings for lower limits. Lower limits will be required in the future so it is imperative that lower levels could be established with these technologies. Obtaining lower levels required an additional phase known as polishing. This phase added used $FeCl_3$ as a polishing agent which resulted in all three technologies producing phosphorus concentrations less than 0.05 mg/L.

For the design of our WWTP values listed in Tables 18-21 all prove useful. Flow rates and plant footprints are different between Westborough's WWTP and our proposed design. Size and flows however are not the concern since the pilot study was conducted under equal operating conditions for all three technologies. Although final costs between the pilot study and our

proposed design may vary, actual values for a smaller plant will be comparable. Smaller plants may require less chemical precipitations, cheaper capital costs, and lower O&M fees. Although since there is a great significance in cost between ACTIFLO™ and the other two technologies it is safe to assume ACTIFLO™ will be the cheapest no matter the plant size.

Design of our WWTP will include ACTIFLO™ into the treatment process. ACTIFLO™ was chosen based on numerous variables. Variables included cost, 20 year projected cost, flexibility, and track record. Assuming that costs and chemical dosages when scaled down for a smaller plant will be comparable; ACTIFLO™ is assumed to be the least expensive. ACTIFLO™ clearly demonstrated in the pilot study to have the lowest cost for a projected 20 years life cycle. Along with its low cost, ACTIFLO™ demonstrated good abilities in using multiple chemicals for precipitation of phosphorus.

Flexibility was evaluated numerous ways. ACTIFLO™ was extremely "flexible" regarding its abilities to precipitate wastewater with multiple chemicals. Along with chemical precipitation ACTIFLO™ can also be easily adapted to existing WWTP's. Westborough's Treatment Facility was able to fit ACTIFLO™ into existing hydraulics avoiding intermediate piping, cutting down on installation costs. Existing sand filters could also be used for additional removal of phosphorous by additional filtering. With a goal of removing as much phosphorus as possible extra filtration makes this technology more appealing.

Veolia Water Solutions & Technologies (VWS) is the water and wastewater specialist that developed the ACTIFLO™ system. VWS has been operating for over 70 years with clients in over 50 countries (Veolia, 2008a). Containing longest and strongest track record of the three technologies studied, ACTIFLO™ is without a doubt the first choice for our design.

Design for the WWTP will include ACTIFLO™ for a phosphorous removal technology. However, all three technologies were sized for design. Calculations for Blue PRO®, CoMag™, and ACTIFLO™ can be found in APPENDIX 1.

6.2 Disinfection tanks

Following the ACTIFLO™ P removal tanks will be disinfection units. Disinfection units will be used to reduce toxins that are present in the wastewater. Three different wastewater disinfection units were analyzed before making a final decision. Factors such as flow accommodations, maintenance, installation, and lifecycle were all considered when analyzing these units.

6.2.1 TrojanUV4000Plus™

Trojan Technologies (2008c) state "the TrojanUV4000Plus™ is the first commercially-successful, medium-pressure UV lamp system that's specifically designed for handling high volume and lower quality wastewater." This disinfection unit features ActiClean™, a full automatic chemical and mechanical cleaning. This process is used to ensure the lamp sleeves are always clean. High-output and low-pressure amalgam lamps will automatically dim based on flow rates and water clarity. Automatic dimming systems save money and energy over long periods of time making the TrojanUV4000Plus™ a cost efficient investment. The TrojanUV4000Plus™ shown in Figure 10 is capable of serving municipalities with populations between 50,000 and 5 million people. Disinfection units can accommodate peak flows of 10 MGD. The system features a contoured reactor and a module removal mechanism making the system virtually maintenance free.

Figure 10: TrojanUV4000Plus™

(Trojan, 2008c)

6.2.2 TrojanUV3000Plus™

The TrojanUV3000Plus™ is one of the most self-sufficient disinfection units available. The TrojanUV3000Plus™, like the TrojanUV4000Plus™ incorporates ActiClean™ therefore making it energy efficient and cost effective. The system accommodates populations between 25,000 and 300,000 people (Trojan, 2008d). Flow accommodations range from 5 to 20 MGD. Figure 11 shows the TrojanUV3000Plus™.

Figure 11: TrojanUV3000Plus™

(Trojan, 2008d)

6.2.3 TrojanUV3000PTP™

The TrojanUV3000PTP™ shown in Figure 12 is the smallest disinfection system designed by Trojan Technologies. Accommodating flows up to 1.4 MGD it is a generic type of UV disinfection (Trojan, 2008b). The price is competitive to similar units and ideal for low budget plants. This system uses low-pressure and low output lamps making it energy efficient although it requires manual cleaning. If the system is not properly maintained it will in turn not operate properly and possibly incur many more expenses not associated wit the TrojanUV3000Plus™ or TrojanUV4000Plus™.

Figure 12: TrojanUV3000PTP™

(Trojan, 2008b)

6.3 WWTP Disinfection Tank

After reviewing the three disinfection tanks the team decided to install the TrojanUV3000Plus™ for the following reasons. For the WWTP with a flow of 3 MGD would require three TrojanUV3000PTP™ disinfection tanks. Three tanks that would all require maintenance was not appealing to the team. The TrojanUV4000Plus™ had the ability to meet the flow requirements of the WWTP although the system is extremely expensive and larger than required. Out of the three disinfection units the TrojanUV3000Plus™ meet the requirements that

our WWTP needed to fulfil. It can accommodate the 3 MGD and with the ActiClean™ system proves to be time and energy efficient. One TrojanUV3000Plus™ will be installed directly following the ACTIFLO™ tanks installed on each train. All effluent flows from the ACTIFLO™ tanks will be funnelled into one train into the TrojanUV3000Plus™.

7.0 Recommendations for Final Design

After considering various ways to design all aspects of the WWTP the proportions for our design were calculated. The final design incorporates flow rates for each train, amounts of BOD_5 (mg/L), and phosphorous reduction optimization. Specific aspects of the WWTP that were designed consists of both Primary and Secondary Clarifiers, Trickling filters, Rotating Biological Contactors, Aeration Tanks, Phosphorous Removal Technology, and a Complete-Mix Activated Sludge System.

The Final Design is illustrated as a top view below in Figures 13 and 14. Figure 13 labeled Design Drawing 1 shows the general design of the WWTP before the addition of a phosphorous removal technology and specified UV disinfection tank. WWTP Design Drawing 1 contains Primary and Secondary Clarifiers, Trickling Filters, Aeration Tanks and the previously selected Complete-Mix Activated Sludge system (Design 2). Other labeled images in both Figures 13 and 14 were not designed but have been inserted to illustrate the entire wastewater treatment system. Figure 14 labeled Design Drawing 3 shows the upgraded WWTP including the UV disinfection tank and Actiflo™ unit. For both figures red lines illustrate flow and direction of wasted sludge. Also both figures display blue lines to demonstrate effluent flow and direction. A

more detailed figure of Actiflo™ unit chosen for the WWTP can be found in Figure 15, Chapter 7 section 4 (7.4).

Figure 13: Design Drawing 1

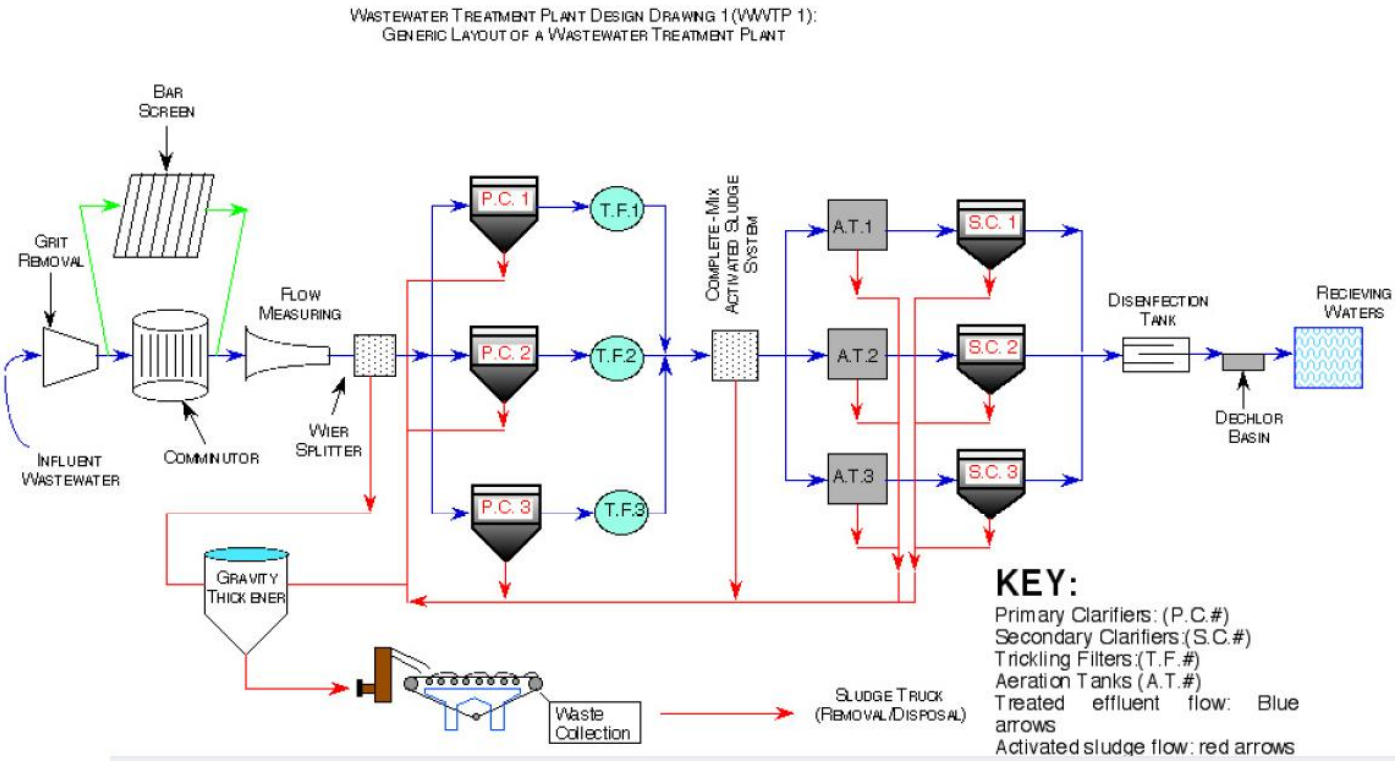
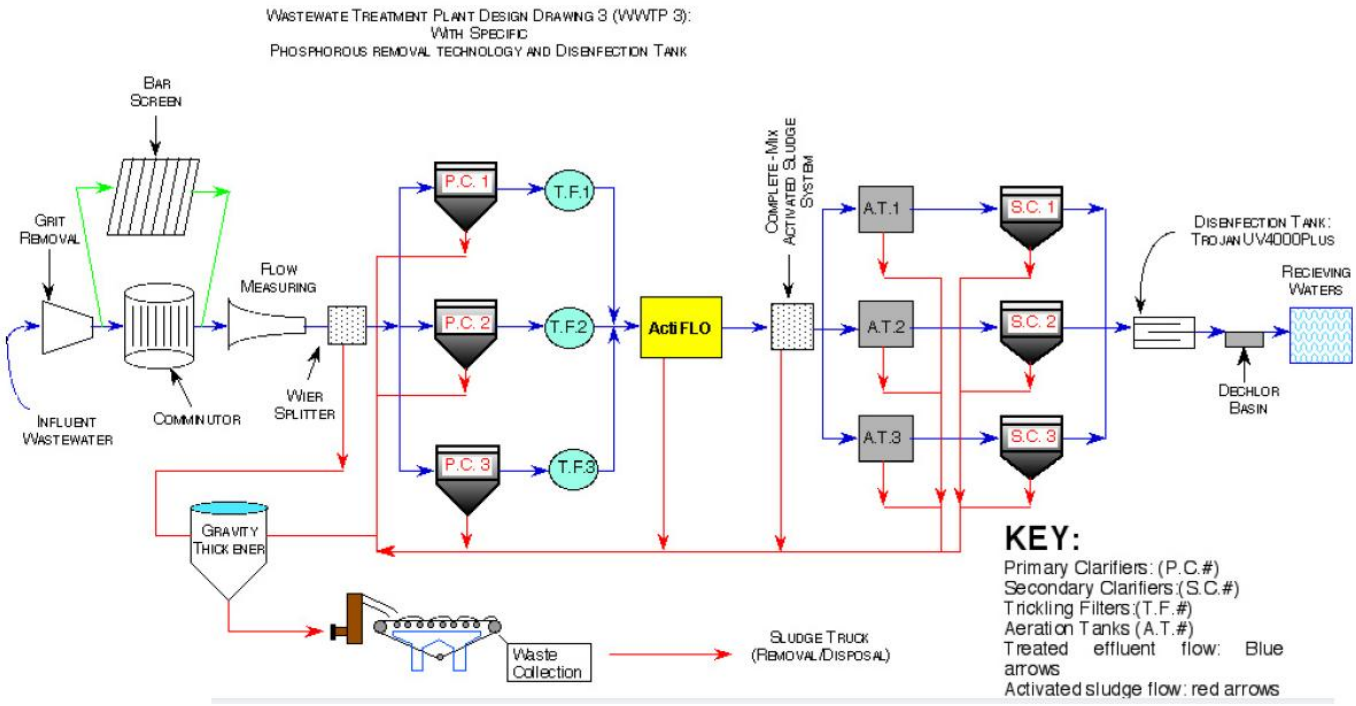


Figure 14 Design Drawing 3



7.1 Primary Clarifiers

As mention before the Primary Clarifier's surface area was calculated based on flow and overflow rates. Table 22 shows the Minimum surface area allowed to meet the flow and overflow rates. This value was obtained from calculations presented in Appendix A. For construction purposes this value is not realistic. Recommend dimensions for fabrication are listed adjacent the calculated surface area in Table 22. The dimensions yield a larger surface area allowing the Primary Clarifiers to accept larger influent flow rates.

Table 22: Primary Clarifier

Minimum Surface Area Allowed	Recommended Design Dimensions			
ft^2	Tank Radius (ft)	Surface Area (ft^2)	Depth (ft)	Volume (ft^3)
1,667	25	1963.5	15	29,452.5

7.2 Trickling Filters

High rate, single stage Trickling Filters were designed to take effluent from Primary Clarifiers. Below in Table 23 minimum volumes and areas are displayed. Calculations for volume and area to satisfy design requirements can be found in Appendix A. Similar issues are presented with these calculated minimum values as seen in Primary Clarifier design. Dimensions to obtain the area and volume are not easily fabricated. Recommended dimensions for the Trickling Filters depth and radius are listed below. Resulting surface area and volume based on new recommended dimensions can also be found in Table 23.

Again, overall surface area exhibits an increase for the recommended dimensions. Similar to P.C. design, larger surface area and volume give the plant more flexibility when treating high flow periods.

Table 23: Trickling Filter Design Specs.

Required Minimum			Recommended Dimensions			
Volume ft^3	Area ft^2	acres	Depth (ft)	Radius of Tank (ft)	Surface Area (ft^2)	Volume (ft^3)
36,167	7,233.4	0.17	5	50	7,854	39,270

7.3 Complete-Mix Activated Sludge

Design for a Complete-Mix Activated Sludge system was performed twice. All calculations for both designs are located in Appendix A and B. Design 2 was selected by the team as the recommended layout. The design was selected based on safety factors. Design 2 is constructed to treat higher levels of MLSS and MLVSS entering the plant. Having the capability of treating more contaminated wastewater less strain will be placed on the system during high flow periods.

The team also believes phosphorous levels can be easier maintained in these high flow conditions due to the larger systems capability of handing more wastewater. Table 24 illustrates the calculated minimum volume for the activated sludge reactor. Recommended dimensions next to minimum volumes have been sized up for construction purposes.

Table 24: Activated Sludge System Design Specs

Required Minimum		Recommended Dimension for Design				
Volume		Volume		Length	Depth	Width
Gallons	ft^3	gallons	ft^3	ft	ft	ft
1,270,000	169,786	1,683,000	225,000	150	15	100

7.4 ACTIFLO™:

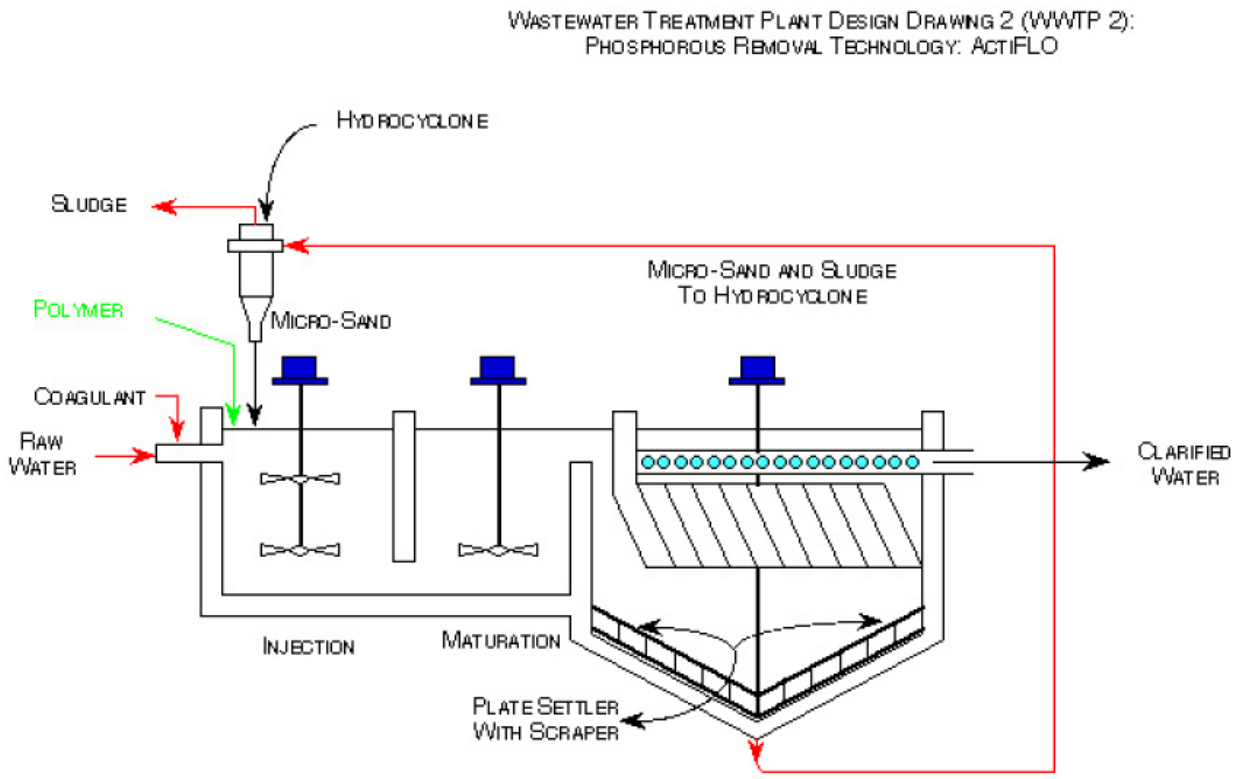
Calculated quantities for the ACTIFLO™ tank dimensions have been listed below in Table 25. Performed calculations can be found in Appendix D. This tank was designed to accommodate 3 trains of 1.0 MGD each. Our team recommends the tank is placed immediately after the Trickling Filters, with effluent from all three flowing into the unit. Effluent from the ACTIFLO™ tank will then flow into a complete mix activated sludge system before entering secondary treatment. Again required minimum values have been listed adjacent to the recommended dimensions established by our team.

Table 25: ACTIFLO™ Tank Dimensions

Required Minimum		Recommended Dimensions for Design				
Surface Area ft^2	Volume ft^3	Length (ft)	Width (ft)	Height (ft)	Surface Area (ft^2)	Volume (ft^3)
125	1,700	15	15	10	225	2250

Figure 15 below as stated above is a more detailed drawing of the Actiflo™ system to be installed. This figure demonstrates the wastewater flow through the system as described in Chapter 4.

Figure 15: Design Drawing 2



7.5 Aeration Tanks

Effluent from the phosphorous removal technology will enter the designed Aeration Tanks. Table 26 shows the recommended detention period for wastewater. A range of 6-7.5 hours is mandatory for the aeration tank. To ensure the tank would be able to treat high flow periods the team designed for a 6 hour detention period (required minimum). However, longer detention periods may be used during lower flowing situations to produce a higher quality effluent. The minimum required volume (ft^3) displayed below was calculated in Appendix C. Again new recommended dimensions for construction have been listed in Table 26.

Table 26: Aeration Tank Design Specs.

Required Minimum	Recommended Dimensions for Design			
Volume	Volume	Length	Width	Depth
ft^3	ft^3	ft	ft	ft
20,143	25,000	100	50	5

7.6 Secondary Clarifier

Calculating a Secondary Clarifiers surface area, unlike P.C. use recirculation flow, along with flow and overflow rates. Table 27 displays this value, labeled as the minimum surface area allowed. Calculations performed to obtain this value can be found in Appendix A. Adjacent to the minimum surface area is again the recommended design dimensions.

Table 27: Primary Clarifier

Required Minimum	Recommended Design Dimensions			
Surface Area ft^2	Tank Radius (ft)	Surface Area (ft^2)	Depth (ft)	Volume (ft^3)
1,667	25	1963.5	15	29,452.5

7.7 Rotating Biological Contactor

Design for the Rotating Biological Contactor required three stages. Table 28 shows a calculated (Appendix A) value for the first stage minimum required area to be 217,000 ft^2. First stage design is half of the total area resulting in both stages 2 and 3 equaling ¼ of the total area. This value was rounded up to 250,000 ft^2 as shown below allowing second and third stages to be easily constructed.

Table 28: Rotating Biological Contactor Design Specs.

Surface media (Area)	Total Area	First Stage	Second Stage	Third Stage
ft^2	ft^2	ft^2	ft^2	ft^2
543,000	---	217,000	---	---
Recommended Design Values				
543,000	550,000	250,000	150,000	150,000

References

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Appendix

Appendix A: WWTP Design Parameters

$$Bod_5\ Influent: 200\ mg/l$$

$$3\ Parallel\ Trains\ of\ 1.0\ MGD\ each$$

Conventional Activated Sludge

Aeration Tank (6 hour detention period):

$$6\ hours\ x\ \frac{1\ day}{24\ hours} = 0.25\ day$$

$$Hydraulic\ loading\ volume = (1.0\ \tfrac{mg}{day})(0.25\ day) = 250{,}000\ gallons$$

$$= \frac{250{,}000\ gallons}{7.48\ {gallons}/{ft^3}}$$

$$= 33{,}442\ ft^3$$

Organic Loading:

$${lbs}/{day} = conc.({mg}/{l})\ x\ Flow\ (MGD)\ x\ 8.34\ {lbs}/{gallon}$$

$${lbs\ BOD_5}/{day} = 200\ {mg}/{litre}\ x\ 1.0MGD\ x\ 8.34\ {lbs}/{gallon} = 1{,}668\ lbs/day$$

Removing 35 % of BOD_5 in primary clarifier:

$$1{,}668\frac{lbs}{day}(1.0 - 0.35)$$

$$= 1{,}084.2\ {}^{lbs}/_{day}\ (0.65) = 705\ {}^{lbs}/_{day}$$

- ✓ **The aeration tank will receive $\mathbf{705\ lbs\ /\ day\ of\ Bod_5}$**

$$Volume\ = \frac{\left(705\ {}^{lbs}/_{day}\right)}{35\ \frac{lbs}{day\ 1{,}000\ ft^3}}$$

$$= 20.143\ x\ 1{,}000 ft^3 = 20{,}143 ft^3$$

$$=\ 20{,}143 ft^3\ x\ 7.48\ {}^{gallons}/_{ft^3}$$

$$= 150{,}669\ gallons$$

- ✓ **BOD_5 Loading is less than hydraulic loading volume (HLC) which is = 33,442 ft^3**

Primary and Secondary clarifier

$$Flow\ = \frac{1.0\ {}^{mg}/_{day}}{600\ {}^{gallons}/_{day - ft^2}}$$

$$Surface\ Area = \frac{Flow}{Oveflow\ Rate}$$

$$= 1{,}667\ ft^2\ of\ surface\ area$$

Trickling filter:

Organic Loading with a primary clarifier:

$$Lb\ BOD/day\ =\ (200\ mg/l)(8.34)(1.0 mgd)(0.65)$$

$$= 1{,}085\ lb\ BOD/day$$

- ✓ **Use 30 lb BOD/day-1,000 ft^3**

$$Volume = {}^{1{,}085 lb/day}\Big/_{30.0\ lb/day - 1{,}000\ ft^3}$$

$$= 36{,}167\ ft^3$$

With a depth of 5 feet:

$$Area = {}^{36{,}167\ ft^3}\Big/_{5.0\ ft} = 7{,}233.4\ ft^2$$

$$= {}^{7{,}233.4\ ft^2}\Big/_{43{,}560\ ft^2/_{acre}} = 0.17\ acre$$

Without recirculation:

$$Hydraulic\ loading = {}^{1.0 mgd}/_{0.17\ acre} = \frac{5.9\ mg}{acre - day}$$

Range is low for high rate (10-30, which is being used for design) and high for low rate (2-5)

- With a recycle ratio of 1, flow is 2.0mgd
- With a recycle ratio of 2, flow is 3.0 MGD

$$\boldsymbol{r} = \mathbf{1}: 2.0/0.17 = 11.76\ mg/acre - day$$

$$\boldsymbol{r} = \mathbf{2}: 3.0/0.17 = 17.64\ mg/acre - day$$

✓ ***We will use the r = 1: = 11.76 mg/acre-day***

Primary clarifier is designed based on Flow/Overflow rate

$$Area = {}^{\frac{1{,}000{,}000\ gal}{day}}\Big/_{\frac{600\ gal}{day - ft^2}} = 1{,}667\ ft^2$$

Secondary clarifier is designed based on Flow + Recirculation Flow / Overflow rate

$$Area = \frac{2{,}000{,}000\ gal/day}{600\ gal/day - ft^2} = 3{,}334\ ft^2$$

Rotating Biological Contactor:

$$RBC\ with\ Primary\ Clarifier: : Lb\ BOD/day = 1085\ lb\ BOD/day$$

Design for organic loading with overall area:

$$Surface\ media = \frac{1085\ lb\ BOD/day}{2\ lb\ BOD/day - 1000 ft^2}$$

$$= 543 - 1000\ ft^2$$

$$= 543{,}000\ ft^2$$

$$First\ stage = \frac{1085 lb\ BOD/day}{5\ lb\ BOD/day - 1000 ft^2} = 217 - 1000 ft^2 = 217{,}000\ ft^2$$

$$Total\ Area = 550{,}000\ ft^2$$

$$1st\ stage = 250{,}000\ ft^2$$

$$2nd\ stage = 150{,}000\ ft^2$$

$$3rd\ stage = 150{,}000 ft^2$$

Appendix B: Complete-Mix Activated Sludge System Design 1

Design one was based on a similar design found in Metcalf (1991). Complete-Mix Activated Sludge System with a flow of 3.0 MGD containing 200 mg/L of BOD_5. Effluent will contain 5 mg/L of BOD_5 or less with the following conditions applying to the wastewater.

- Temperature=20^{o}C
- Influent volatile suspended solids to reactor are negligible

- Ratio of mixed-liquor volatile suspended solids (MLVSS) to mixed-liquor suspended solids (MLSS) = 0.8
- MLVSS = 1,840 mg/L
- MLSS = 2,300 mg/L
- Return sludge concentration = 5,254 mg/L
- Design mean cell-residence time θ_c = 10 d
- Effluent contains 18 mg/L of biological solids of which 60 percent is biodegradable
- $BOD_5 = 0.68 \times BOD_L$
- 1.42 mg O_2 Is consumed per mg of cell oxidized
- MLSS settling data taken from a plant pilot study (table 22)

Table 29: MLSS Settling Data

(Metcalf, 1991)

MLSS, mg/L	1,600	2,500	2,600	4,000	5,000	8,000
Initial settling, velocity, ft/h	11.0	8.0	5.0	2.0	1.0	0.3

1. Estimated concentration of BOD_5 in the effluent using the following relationship:

   Effluent BOD_5 = influent soluble BOD_5 escaping treatment + BOD_5 of effluent suspended solids

   - BOD_5 of effluent suspended solids
     i. Biodegradable portion of effluent biological solids is:

     $= \%\ of\ solids\ biodegradable\ x\ effluent\ biological\ solids\ \frac{mg}{L}$

$$0.60(18\,mg/L) = 10.8\,mg/L$$

ii. Ultimate BOD_L of the biodegradable effluent solids is:

$$= \left[0.6\left(18\frac{mg}{L}\right)\right]\left(1.42\ \frac{mg\ of\ O_2 consumed}{mg} cell\ oxidized\right)$$

$$= 15.34\frac{mg}{L}$$

iii. BOD_5 of effluent suspended solids:

$$15.34\frac{mg}{L}(0.68) = 10.43\,mg/L$$

- Influent soluble BOD_5 escaping treatment

$$20\frac{mg}{L} = S + 10.34\frac{mg}{L}$$

$$S = 9.57\frac{mg}{L}$$

2. Treatment Efficiency E:

$$E = [(S_0 - S)/S]\ x\ 100$$

i. E = process efficiency

ii. S_o = influent substrate concentration

iii. S = effluent substrate concentration

- Efficiency based on soluble BOD_5 denoted E_s:

$$E_s = \frac{(BOD_5 - S)mg/L}{BOD_5\frac{mg}{L}}x100$$

$$E_s = \frac{(200 - 9.57)mg/L}{200\frac{mg}{L}}x100 = 95.2\%$$

- The overall plant efficiency or $E_{overall}$:

$$E_{overall} = \frac{(BOD_5 - BOD_5\ escaping\ treatment)mg/L}{BOD_5\frac{mg}{L}}x100$$

$$E_{overall} = \frac{(200 - 20)\ mg/L}{200\frac{mg}{L}} x100 = 90.0\%$$

3. Reactor Volume:

$$X = \frac{\theta_c Y(S_o - S)}{\theta(1 + k_d\theta_c)}$$

$$\theta = \frac{V_r}{Q}$$

i. X = mass concentration of microorganisms

ii. Θ_c = detention time

iii. K_d = kinetic coefficient

iv. V_r = volume of the reactor

v. Q = WWTP flow measured in MGD

vi. Θ = hydraulic detention time (V/Q)

- Combining two equations and solving for V_r:

$$V_r = \frac{\theta_c QY(S_o - S)}{X(1 + k_d\theta_c)}$$

- $\theta_c = 10\ d$
- $Q = 3\ MGD$
- $Y = 0.5\frac{lb}{lb}\ (Metcalf, 1991, table\ 8-7\ page\ 394)$
- $S_o = 200\frac{mg}{l}$
- $S = 9.57\ mg/L$
- $X = 1{,}840\ mg/L$
- $k_d = 0.06d^{-1}(table\ 8-7)$

$$V_r = \frac{(10\ d)(3MGD)(0.50)[(200 - 9.57)\frac{mg}{L}]}{\left(1840\frac{mg}{L}\right)(1 + 0.06\ x\ 10)}$$

$$= 0.97\ Million\ gallons$$

4. Quantity of sludge wasted every day

- Y_{obs} = observed yield with recycle

$$Y_{obs} = \frac{Y}{1 + k_d\theta_c}$$

$$= \frac{0.5}{(1 + 0.06\ x\ 10)}$$

$$= 0.3125$$

- Increase of mass of MLVSS or P_x:

$$P_x = Y_{obs}\,Q(S_o - S)(8.34)$$

  - 8.34 = conversion factor [lb/Mgal·(mg/L)]

$$= 0.3125(3MGD)\left(200 - 9.57\frac{mg}{L}\right)[8.34\ lb/Mgal \cdot (\frac{mg}{L})$$

$$= 1{,}488.92\ lb/d$$

- Increase in total mass of MLSS denoted $P_{x(ss)}$:

$$P_{x(ss)} = \frac{P_x}{(MLVSS/MLSS)}$$

$$P_{x(ss)} = \frac{1{,}488.92\ lb/d}{0.8}$$

$$= 1{,}861.2\frac{lb}{d}$$

- Sludge to be wasted

Mass wasted = increase in MLSS – SS lost in effluent

$$= 1{,}861.2\,\frac{lb}{d} - \ 3MGD\ x\ 18\,\frac{mg}{L}\,x\left[8.34\,\frac{lb}{Mgal\cdot\left(\frac{mg}{L}\right)}\right]$$

$$= 1{,}411.92\ lb/d$$

5. Sludge wasting rate if wasting is accomplished from the reactor assuming $Q_e = Q$ and VSS in effluent is equal to 80% of the SS

$$\theta_c = \frac{V_r X}{Q_w X + Q_e X_e}$$

   - Q_w = flow rate of liquid containing wasted biological cells
   - Q_e = separation unit flow rate
   - X_e = separation unit effluent concentration of microorganism

$$10d = \frac{(0.97Mgal)(1{,}840\,\frac{mg}{L})}{\left(\frac{Q_w Mgal}{d}\right)\left(1{,}840\,\frac{mg}{L}\right) + \left(\frac{3Mgal}{d}\right)(18\,\frac{mg}{L}\,x\ 0.8)}$$

$$Q_w = 0.078\ Mgal/day$$

6. Recirculation ratio writing a mass balance around the reactor:

   - Aerator VSS concentration = 1,840 mg/L
   - Return sludge concentration = 5,254 mg/L
   - VSS concentration = Return sludge concentration (mg/L) x (MLVSS/MLSS)
   - Return VSS concentration = 5,254 x 0.8
   - VSS =4,203.2 mg/L

$$(1{,}840\ mg/L)\cdot\ (Q + Q_r) = \ (Q_r)4{,}203.2\ mg/L$$

$$[1{,}840(Q) + \ 1{,}840(Q_r)]mg/L(= (Q_r)4{,}203.2\ mg/L$$

$$[1{,}840(Q)]mg/L = (Q_r)2{,}363.2\ mg/L$$

$$1{,}840\ mg/L \cdot (3MGD) = (Q_r)2{,}363.2\ mg/L$$

$$Q_r = 2.34$$

$$\frac{Q_r}{Q} = \alpha$$

$$\alpha = \frac{2.34MGD}{3\ MGD}$$

$$\alpha = 0.78$$

7. Hydraulic retention time for the reactor :

$$\theta = \frac{V_r}{Q}$$

$$= \frac{0.97\ Mgal}{(3MGD)} = 0.32\ d$$

$$0.33\ d\ x \frac{24hr}{d} = 7.76\ hr$$

8. Oxygen requirements based on ultimate carbonaceous demand, BOD_L

- Mass of ultimate BOD_L of incoming wastewater that is converted in the process assuming that the BOD_5 is equal to 0.68 BOD_L.

$$Mass\ of\ BOD_L\ utilized = \frac{Q[(S_o - S)mg/L]}{0.68}\ x\ 8.34$$

$$= \frac{3\ MGD\ (200\frac{mg}{L} - 9.57\frac{mg}{L})}{0.68} \cdot \frac{8.34\ lb}{Mgal\ x\ (\frac{mg}{L})}$$

$$= 7{,}006.7\frac{lb}{d}$$

- Oxygen Requirement per day

$$lb\ of\ O_2/d = \frac{Q(S_o - S)x\ 8.34}{f} - 1.42(P_x)$$

i. f = conversion factor for BOD_5 to BOD_L (0.68)

$$lb\ of\ \ O_2/d = 7{,}006.7\frac{lb}{d} - 1.42(1{,}488.92)\ lb/d$$

$$= 4{,}892.43\frac{lb}{d}$$

9. Food to microorganism (F/M) ratio and volumetric loading factor

- F/M ratio equation

$$\frac{F}{M} = \frac{S_o}{\theta X} = \frac{200\frac{mg}{l}}{(0.32\ d)\left(1{,}840\frac{mg}{L}\right)}$$

$$= 0.34\ d^{-1}$$

- Volumetric Loading

$$Volumetric\ Loading, \frac{lb}{10^3 ft^3}\ \cdot\ d = \frac{S_o Q}{V_r} \cdot 8.34 \cdot (\frac{1{,}000}{10^3})$$

$$= \frac{\left(200\frac{mg}{L}\right)(3MGD)}{\frac{97{,}000\ gal}{\frac{7.48 gal}{ft^3}}} \cdot \left[8.34\frac{lb}{Mgal} \cdot \left(\frac{mg}{L}\right)\right]\left(\frac{1{,}000 ft^3}{10^3 ft^3}\right)$$

$$= 385\ lb\ BOD_5/10 ft^3 \cdot d$$

10. Volume of air required. Oxygen transfer efficiency assumed to be 80% with a safety factor of 2 used for design of blowers.

- Theoretical air requirement for air containing 23.2% oxygen by weight

$$Theoretical\ air\ requirement = \frac{(lb\ of\ \ O_2/d)}{(0.075\frac{lb}{ft^3} x 0.232)}$$

$$\frac{4{,}892.43\frac{lb}{d}}{\left(0.075\ {}^{lb}/_{ft^3}\right)(0.232)} = 281{,}174\ {}^{ft^3}/_{d}$$

- Air requirement at 8% transfer efficiency

$$Air\ requirement\ at\ 8\% = \frac{Theoretical\ air\ requirement}{transfer\ efficency}$$

$$\left(\frac{284{,}174}{0.08}\right) ft^3/_d = 3{,}514{,}676\, ft^3/_d$$

$$\frac{3{,}514{,}676\, ft^3/_d}{1{,}440\, min/_d} = 2{,}441\, ft^3/_{min}$$

- Design air requirement

$$2\left(2{,}441\, ft^3/_{min}\right) = 4{,}882\, ft^3/_{min}$$

11. Volume Check

- Air requirement per unit volume

$$Air\ per\ unit\ volume = {Air\ requirement\ at\ 8\ \%}/{Q}$$

$$\frac{3{,}514{,}676\, ft^3/_d}{3{,}000{,}000\, gal/_d} = 1.17\, ft^3/_{gal}$$

- Air requirement per lb of BOD_5 removed

$$Air\ requirement\ per\ lb\ of\ B0D_5 = \frac{Air\ requirement\ at\ 8\ \%}{(S_o - S)(Q)(8.34)}$$

$$\frac{3{,}514{,}676\, ft^3/_d}{\left(200\, mg/_l - 9.57\, mg/_l\right)\left(3Mgal/_d\right)(8.34\, lb/_{Mgal} \cdot \left(\frac{mg}{l}\right))}$$

$$= 737.67\, ft^3/_{lb\ of\ BOD_5\ removed}$$

12. Recycle Ratio required to maintain mixed-liquor suspended-solids concentration at:

$$Ratio = \frac{MLVSS}{(MLVSS/MLSS)}$$

$$\frac{1{,}840\ mg/L}{0.8} = 1{,}472\ {}^{mg}/_{l}$$

- Materials balance on influent to the reactor

$$Q(X_o) + Q_r(X_u) = (Q + Q_r)\ x\ 1{,}472\ {}^{mg}/_{l}$$

$$Q = influent\ flow\ MGD$$

$$Q_r = recycle\ flow\ rate\ MGD$$

$$X_o = influent\ suspended\ solids\ ({}^{mg}/_{l})$$

$$X_u = underflow\ suspended\ solids\ ({}^{mg}/_{l})$$

$$Assuming\ X_o = 0\ and\ Q_r = (\propto \cdot Q)\ we\ rewrite\ the\ above\ equation:$$

$$\propto QX_u - \propto \left(1{,}472\ {}^{mg}/_{l}\right)Q = Q\left(1{,}472\ {}^{mg}/_{l}\right)$$

$$\propto = \frac{1{,}472\ {}^{mg}/_{l}}{X_u\ {}^{mg}/_{l} - 1{,}472\ {}^{mg}/_{l}}$$

$$\propto = recycle\ ratio\ {}^{Q_r}/_{Q}$$

Recycle ratios for various underflow conditions shown below in Table 5.

Table 30: Recycle Ratios

X_u, mg/l	2,000	3,000	3,500	4,000	5,000
X_u - 1472 (mg/l)	528	1,528	2,026	2,528	3,528

α	2.78	0.96	0.73	0.58	0.42

Appendix C: Complete-Mix Activated Sludge System Design 2

Facility treats 3.0 MGD containing 200 mg/L of BOD_5. The effluent will have 5 mg/L of BOD_5 or less with the following conditions applying to the wastewater.

- Temperature=20^oC
- Influent volatile suspended solids to reactor are negligible
- Ratio of mixed-liquor volatile suspended solids (MLVSS) to mixed-liquor suspended solids (MLSS) = 0.63
- MLVSS = 2,200 mg/L
- MLSS = 3,500 mg/L
- Return sludge concentration = 5,254 mg/L
- Design mean cell-residence time θ_c = 24 d
- Effluent contains 18 mg/L of biological solids of which 60 percent is biodegradable
- BOD_5 = 0.68 x BOD_L
- 1.42 mg O_2 Is consumed per mg of cell oxidized
- MLSS settling data taken from a plant pilot study (Table 24)

Table 31: MLSS Settling Data

(Metcalf, 1991)

MLSS, mg/L	1,600	2,500	2,600	4,000	5,000	8,000
Initial settling, velocity, ft/h	11.0	8.0	5.0	2.0	1.0	0.3

13. Estimated concentration of BOD_5 in the effluent using the following relationship:

Effluent BOD_5 = influent soluble BOD_5 escaping treatment + BOD_5 of effluent suspended solids

- BOD_5 of effluent suspended solids

iv. Biodegradable portion of effluent biological solids is:

$$= \%\ of\ solids\ biodegradable\ x\ effluent\ biological\ solids\ \frac{mg}{L}$$

$$0.60(18\ mg/L)\ =\ 10.8\ mg/L$$

v. Ultimate BOD_L of the biodegradable effluent solids is:

$$= \left[0.6\left(18\frac{mg}{L}\right)\right]\left(1.42\ \frac{mg\ of\ O_2 consumed}{mg}\ cell\ oxidized\right)$$

$$=\ 15.34\frac{mg}{L}$$

vi. BOD_5 of effluent suspended solids:

$$15.34\frac{mg}{L}(0.68) = 10.43\ mg/L$$

- Influent soluble BOD_5 escaping treatment, S:

$$20\frac{mg}{L} = S + 10.34\frac{mg}{L}$$

$$S = 9.57\frac{mg}{L}$$

14. Treatment Efficiency E:

$$E = [(S_0 - S)/S]\ x\ 100$$

i. E = process efficiency

ii. S_o = influent substrate concentration

iii. S = effluent substrate concentration

- Efficiency based on soluble BOD_5 denoted E_s:

$$E_s = \frac{(BOD_5 - S)mg/L}{BOD_5 \frac{mg}{L}} x100$$

$$E_s = \frac{(200 - 9.57)mg/L}{200 \frac{mg}{L}} x100 = 95.2\%$$

- The overall plant efficiency or $E_{overall}$:

$$E_{overall} = \frac{(BOD_5 - BOD_5\ escaping\ treatment)mg/L}{BOD_5 \frac{mg}{L}} x100$$

$$E_{overall} = \frac{(200 - 20)\ mg/L}{200 \frac{mg}{L}} x100 = 90.0\%$$

15. Reactor Volume:

$$X = \frac{\theta_c Y(S_o - S)}{\theta(1 + k_d \theta_c)}$$

$$\theta = \frac{V_r}{Q}$$

i. X = mass concentration of microorganisms

ii. Θ_c = detention time

iii. K_d = kinetic coefficient

iv. V_r = volume of the reactor

v. Q = WWTP flow measured in MGD

vi. Θ = hydraulic detention time (V/Q)

- Combining two equations and solving for V_r:

$$V_r = \frac{\theta_c QY(S_o - S)}{X(1 + k_d \theta_c)}$$

- $\theta_c = 24\ d$
- $Q = 3\ MGD$
- $Y = 0.5\frac{lb}{lb}\ (Metcalf, 1991, table\ 8-7\ page\ 394)$
- $S_o = 200\frac{mg}{l}$
- $S = 9.57\ mg/L$
- $X = 2{,}200\ mg/L$
- $k_d = 0.06 d^{-1} (table\ 8-7)$

$$V_r = \frac{(24\ d)(3MGD)(0.50)[(200 - 9.57)\frac{mg}{L}]}{\left(2{,}200\frac{mg}{L}\right)(1 + 0.06\ x\ 24\ d)}$$

$$= 1.27\ Million\ gallons$$

16. Quantity of sludge wasted every day

- Y_{obs} = observed yield with recycle

$$Y_{obs} = \frac{Y}{1 + k_d\theta_c}$$

$$= \frac{0.5}{(1 + 0.06\ x\ 24\ d)}$$

$$= 0.205$$

- Increase of mass of MLVSS or P_x:

$$P_x = Y_{obs}\, Q(S_o - S)(8.34)$$

  - 8.34 = conversion factor [lb/Mgal·(mg/L)]

$$= 0.205(3MGD)\left(200 - 9.57\frac{mg}{L}\right)[8.34\ lb/Mgal \cdot (\frac{mg}{L})$$

$$= 976.74\ lb/d$$

- Increase in total mass of MLSS denoted $P_{x(ss)}$:

$$P_{x(ss)} = \frac{P_x}{(MLVSS/MLSS)}$$

$$P_{x(ss)} = \frac{976.74\ lb/d}{0.63}$$

$$= 1{,}550.38\frac{lb}{d}$$

- Sludge to be wasted

Mass wasted = increase in MLSS – SS lost in effluent

$$= 1{,}550.38\frac{lb}{d} - \ 3MGD\ x\ 18\frac{mg}{L} x \left[8.34\frac{lb}{Mgal \cdot \left(\frac{mg}{L}\right)}\right]$$

$$= 1{,}100.02\ lb/d$$

17. Sludge wasting rate if wasting is accomplished from the reactor assuming $Q_e = Q$ and VSS in effluent is equal to 80% of the SS

$$\theta_c = \frac{V_r X}{Q_w X + Q_e X_e}$$

- Q_w = flow rate of liquid containing wasted biological cells
- Q_e = separation unit flow rate
- X_e = separation unit effluent concentration of microorganism

$$24\ d = \frac{(0.97Mgal)(2{,}200\frac{mg}{L})}{\left(\frac{Q_w Mgal}{d}\right)\left(2{,}200\frac{mg}{L}\right) + \left(\frac{3Mgal}{d}\right)(18\frac{mg}{L} x\ 0.8)}$$

$$Q_w = 0.027\ \text{Mgal/day}$$

18. Recirculation ratio writing a mass balance around the reactor:

- Aerator VSS concentration = 2,200 mg/L
- Return sludge concentration = 5,254 mg/L

- VSS concentration = Return sludge concentration (mg/L) x (MLVSS/MLSS)
- Return VSS concentration = 5,254 x 0.63 = 3310.02
- VSS =3,310.02 mg/L

$$(2{,}200\ mg/L) \cdot\ (Q + Q_r) =\ (Q_r)3{,}310.02 mg/L$$

$$[2{,}200(Q) +\ 2{,}200(Q_r)]mg/L = (Q_r)3{,}310.02\ mg/L$$

$$[2{,}200(Q)]mg/L = (Q_r)1{,}110.02\ mg/L$$

$$2{,}200\ mg/L \cdot (3MGD) = (Q_r)1{,}110.02\ mg/L$$

$$Q_r = 5.95$$

$$\frac{Q_r}{Q} = \alpha$$

$$\alpha = \frac{5.95MGD}{3\ MGD}$$

$$\alpha = 1.98$$

19. Hydraulic retention time for the reactor :

$$\theta = \frac{V_r}{Q}$$

$$= \frac{1.27\ Mgal}{(3MGD)} = 0.42\ d$$

$$0.42\ d\ x \frac{24hr}{d} = 10.16\ hr$$

20. Oxygen requirements based on ultimate carbonaceous demand, BOD_L

- Mass of ultimate BOD_L of incoming wastewater that is converted in the process assuming that the BOD_5 is equal to 0.68 BOD_L.

$$Mass\ of\ BOD_L\ utilized = \frac{Q[(S_o - S)mg/L]}{0.68}\ x\ 8.34$$

$$= \frac{3\ MGD\ (200\frac{mg}{L} - 9.57\frac{mg}{L})}{0.68} \cdot \frac{8.34\ lb}{Mgal\ x\ (\frac{mg}{L})}$$

$$= 7{,}006.7\frac{lb}{d}$$

- Oxygen Requirement per day

$$lb\ of\ \ O_2/d = \frac{Q(S_o - S)x\ 8.34}{f} - 1.42(P_x)$$

ii. f = conversion factor for BOD_5 to BOD_L (0.68)

$$lb\ of\ \ O_2/d = 7{,}006.7\frac{lb}{d} - 1.42(976.74\ )\ lb/d$$

$$= 5{,}619.73\frac{lb}{d}$$

21. Food to microorganism (F/M) ratio and volumetric loading factor

- F/M ratio equation

$$\frac{F}{M} = \frac{S_o}{\theta X} = \frac{200\frac{mg}{l}}{(0.42\ d\ )\left(2{,}200\frac{mg}{L}\right)}$$

$$= 0.22\ d^{-1}$$

- Volumetric Loading

$$Volumetric\ Loading, \frac{lb}{10^3 ft^3}\ \cdot\ d = \frac{S_o Q}{V_r} \cdot 8.34 \cdot (\frac{1{,}000}{10^3})$$

$$= \frac{\left(200\frac{mg}{L}\right)(3MGD)}{\frac{1{,}270{,}000\ gal}{\frac{7.48gal}{ft^3}}} \cdot \left[8.34\frac{lb}{Mgal} \cdot \left(\frac{mg}{L}\right)\right]\left(\frac{1{,}000ft^3}{10^3 ft^3}\right)$$

$$= 29.47\ lb\ BOD_5/10ft^3 \cdot d$$

22. Volume of air required. Oxygen transfer efficiency assumed to be 80% with a safety factor of 2 used for design of blowers.

- Theoretical air requirement for air containing 23.2% oxygen by weight

$$Theoretical\ air\ requirement = \frac{(lb\ of\ \ O_2/d)}{(0.075\frac{lb}{ft^3}x0.232)}$$

$$\frac{5{,}619.73\frac{lb}{d}}{\left(0.075\,{}^{lb}/_{ft^3}\right)(0.232)} = 322{,}973\,{}^{ft^3}/_{d}$$

- Air requirement at 8% transfer efficiency

$$Air\ requirement\ at\ 8\% = \frac{Theoretical\ air\ requirement}{transfer\ efficency}$$

$$\left(\frac{322{,}973}{0.08}\right){}^{ft^3}/_{d} = 4{,}037{,}162\,{}^{ft^3}/_{d}$$

$$\frac{4{,}037{,}162\,{}^{ft^3}/_{d}}{1{,}440\,{}^{min}/_{d}} = 2{,}803.6\,{}^{ft^3}/_{min}$$

- Design air requirement

$$2\left(2{,}803.6\,{}^{ft^3}/_{min}\right) = 5{,}607.2\,{}^{ft^3}/_{min}$$

23. Volume Check

- Air requirement per unit volume

$$Air\ per\ unit\ volume = {}^{Air\ requirement\ at\ 8\ \%}/_{Q}$$

$$\frac{4{,}037{,}162\,{}^{ft^3}/_{d}}{3{,}000{,}000\,{}^{gal}/_{d}} = 1.35\,{}^{ft^3}/_{gal}$$

- Air requirement per lb of BOD_5 removed

$$Air\ requirement\ per\ lb\ of\ B0D_5 = \frac{Air\ requirement\ at\ 8\ \%}{(S_o - S)(Q)(8.34)}$$

$$\frac{4{,}037{,}162\ ^{ft^3}/_{d}}{\left(200\ ^{mg}/_{l} - 9.57\ ^{mg}/_{l}\right)\left(3Mgal/_{d}\right)(8.34\ ^{lb}/_{Mgal} \cdot (\frac{mg}{l}))}$$

$$= 847.33\ ^{ft^3}/_{lb}\ of\ BOD_5\ removed$$

24. Recycle Ratio required to maintain mixed-liquor suspended-solids concentration at:

$$Ratio = \frac{MLVSS}{(MLVSS/MLSS)}$$

$$\frac{2{,}200\ mg/L}{0.63} = 3{,}492.06\ ^{mg}/_{l}$$

- Materials balance on influent to the reactor

$$Q(X_o) + Q_r(X_u) = (Q + Q_r)\ x\ 3{,}492.06\ ^{mg}/_{l}$$

$$Q = influent\ flow\ MGD$$

$$Q_r = recycle\ flow\ rate\ MGD$$

$$X_o = influent\ suspended\ solids\ (^{mg}/_{l})$$

$$X_u = underflow\ suspended\ solids\ (^{mg}/_{l})$$

$$Assuming\ X_o = 0\ and\ Q_r = \propto Q\ we\ rewrite\ the\ above\ equation:$$

$$\propto QX_u - \propto \left(3{,}492.06\ ^{mg}/_{l}\right)Q = Q\left(3{,}492.06\ ^{mg}/_{l}\right)$$

$$\propto = \frac{3{,}492.06\ ^{mg}/_{l}}{X_u\ ^{mg}/_{l} - 3{,}492.06\ ^{mg}/_{l}}$$

$$\propto = recycle\ ratio\ {}^{Q_r}/_{Q}$$

Recycle ratios for various underflow conditions shown below in Table 25.

Table 32: Recycle Ratios

X_u, mg/l	4,000	5,000	6,000	7,000	8,000
X_u – 3,492 (mg/l)	508	1,508	2,508	3,508	4,508
α	6.87	2.32	1.39	0.99	0.77

Appendix D: Technology Design Loadings

Tank sizes are designed for 3,000,000 MGD treatment plant. One tank will be installed in immediately after the trickling filters prior to the complete-mix activated sludge system. Table below was taken from NEWEA conference pilot study conducted at the Westborough WWTP.

Table 33: Westborough Pilot Study Design Values

(NEWEA, 2008)

Description	Blue Pro®	CoMag™	Actiflo™
Clarification System			
Flocculation Time (min)	----	10	6
Loading rate (gpm/ft^2)	3.5	4.4	16.8
Area required (ft^2)	300	175	50
Waste Production			
Clarifier waste (gpm)	375	160	140
Percent of Total waste (%)	7.0	3.0	2.6
Solids concentration (%)	0.03	0.8-1.0	0.1-0.5

Blue PRO®

Tank Size:

- Surface Area

$$3{,}000{,}000\ {}^{gallons}/_{day}\ x\ {}^{day}/_{24\ hrs}\ x\ {}^{hr}/_{60\ min}$$

$$= 2083.33\ {}^{gallons}/_{minute}$$

$$2083.33\ {}^{gallons}/_{minute} \Big/ 3.5\ {}^{gallons}/_{min}/_{ft^2} = 595.24\ ft^2\ \approx 600\ ft^2$$

Blue Pro requires 50 square feet per filter:

$$600\ {}^{ft^2}/_{50\ ft^2\ per\ filter} = 12\ filters$$

CoMag™

Tank Size:

- Hydraulic loading volume

$$10\ \text{min detention time}\ x\ {}^{1\ day}/_{24\ hours}\ {}^{1\ hour}/_{60min} = 0.006944\ day$$

$$3{,}000{,}000\ {}^{gallons}/_{day}\ x\ 0.0069\ day = 207{,}000\ gallons$$

$$\frac{207{,}000\ gallons}{7.48\ gallons/ft^3} = 27{,}673.8 \cong 27{,}700\ ft^3$$

- Surface Area

$$2083.33\ {}^{gallons}/_{minute} \Big/ 4.4\ {}^{gallons}/_{min}/_{ft^2} = 473.48\ ft^2\ \approx 475 ft^2$$

ACTIFLO™

Tank Size:

- Hydraulic loading volume

$$6 \text{ min detention time } x \; {}^{1\ day}/_{24\ hours} \, x \, {}^{1\ hour}/_{60 min} = 0.004167\ day$$

$$3{,}000{,}000 \; {}^{gallons}/_{day} \; x \; 0.004167\ day = 12{,}501\ gallons$$

$$\frac{12{,}501 gallons}{7.48\ gallons/ft^3} = 1{,}671 \cong 1{,}700\ ft^3$$

- Surface Area (ft^2)

$$2083.33 \; {}^{gallons}/_{minute} \Big/ 16.8 \; {}^{gallons}/_{{}^{min}/_{ft^2}} = 124\ ft^2 \approx 125\ ft^2$$